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General Joe Wheeler Dam work starts. Cofferdam 1450x400 ft. in Tennessee River above Muscle Shoals from which 300,000 cu.yd. of excavation will be removed for the power house and the first section of the dam.

Simplified Analysis of Indeterminate Frames

Construction Progress on the Golden Gate Bridge

Vibrating Concrete for Pavements

The TVA's Second Dam Building Job

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In the News

ITALY'S new 60-mile electrified rail line, including the great Apennine tunnel, between Bologna and Florence, was opened on April 22. The tunnel, 11½ miles long, is over two miles longer than the St. Gothard tunnel in the Alps. There are 29 other tunnels aggregating about 12 miles in length on the line, which represents an investment of about $100,000,000.

MICHIGAN will vote on April 30 on a state construction program involving about $38,000,000, provided by state bonds to be sold to the federal government. State hospitals and armories get about half of the fund, and grade separations the other half.

SEATTLE has sold to a banking syndicate $4,500,000 of bonds providing for resumption of construction on its Skagit River hydro project.

PWA employed 305,000 during March, with a payroll of about $16,000,000.

BID CHISELING was taken in hand by PWA recently, by ordering all contractors to submit with their bids the names of all subcontractors and their bids.

ACTIVATED SLUDGE is soon to invade the chambers of the U. S. Supreme Court. Milwaukee has announced that it will appeal the patent infringement case brought against it by Activated Sludge, Inc.

OVER 100 miles per hour was the speed attained by the Burlington's stainless steel, diesel-powered streamlined train in exhibition runs at Philadelphia last week. Reasonably tangent track was required.

BILLS have been passed by the New York legislature in the past week providing for a thorough re-examination of public-utility practices and development plans. Municipal ownership is fostered in Governor Lehman's program, and it has been adopted practically in full.

In This Issue

DAM BUILDING and the Tennessee Valley Authority have come to the front of current interest. Two projects—Norris Dam and General Joe Wheeler Dam—are under construction, and, in an address at Chattanooga, April 21, David E. Lilienthal, of the Authority, stated "Four other major dams will probably be approved for early construction." The leading article in this issue describes the construction operations at Wheeler Dam (known also as Dam No. 3), which got under way last November.

VIBRATED CONCRETE for pavements is an outstanding development of recent months. Already experiment and test have produced many data. In the first of two articles, practices and recommendations from Missouri, Michigan and the Bureau of Public Roads are presented in some detail. This is the first published survey of general scope that has become available on the most important change in concrete roadbuilding technique that has occurred in several years

ENGINEERING LITERATURE—which comprises books, pamphlets and reports, domestic and foreign—continues to be produced in steady volume. The monthly review of these works is an important part of the contents of this issue.

THE GOLDEN GATE BRIDGE has been under construction for eighteen months. Much progress has been made, and numerous obstacles have been met and largely overcome. The south pier foundation, presenting one of the most difficult bridge-pier jobs on record, is now the controlling factor in future progress. The construction story, to date, is told in this issue.

A NEW PROCEDURE for analyzing indeterminate frames is based on displacement equations derived from Williot diagrams. Developed by Charles A. Ellis (and used in the design of the towers of the Golden Gate Bridge), the Williot strain method, as it is called, is particularly well adapted to the solution of certain complicated structures. An article in this issue describes and compares it with two other methods now in use.

TRAFFIC-CONTROLLED SIGNALS on state routes in municipalities in Illinois are to be operated under state highway department supervision, as a result of a new law. The department's policy and regulations have just been announced.

Coming Articles

STABILIZATION OF EARTH ROADS with the use of calcium chloride has received great development through experiments and actual trials in highway building and maintenance. For two years the Dow Chemical Co. has been working on this type of road treatment, experimenting with various kinds of soils and proportions of mixtures under various traffic conditions. In a series of four articles, to be published soon, W. R. Collings and L. C. Stewart, of the Dow company, will describe laboratory tests of soils, actual traffic tests of stabilized road sections, construction procedure and cost of stabilization.

WHEN IT'S CONCRETE
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ENGINEERING NEWS-RECORD

New York

April 26, 1934

THE FIRST COFFERDAM for General Joe Wheeler Dam extends 1,400 ft. out into the Tennessee River, which is 6,000 ft. wide at this point. On the far shore the navigation lock is under construction.

The TVA's Second Dam-Building Job

Work on General Joe Wheeler Dam and power house on the Tennessee River above Muscle Shoals, begun in late November, now well under way—Large crib cofferdam completed—Electric shovels floated into cofferdam and unloaded after the cofferdam had been unwatered

By W. M. Hall

Tennessee Valley Authority, Engineer in Charge of Construction, General Joe Wheeler Dam

WHILE both Norris Dam on the Clinch River and General Joe Wheeler Dam on the Tennessee —the two dams which the Tennessee Valley Authority is now building—have certain points in common, there is considerable difference in the initial engineering problems involved in their construction. Both dams are intended for the impounding of floodwaters and for the generation of electric power. The General Joe Wheeler Dam will serve the additional purpose of improving navigation over the upper reaches of the Muscle Shoals, and for that reason will be equipped with locks. Both dams are designed to be of the gravity type and of concrete construction.

But there the similarity ends. Norris Dam is in a wild and sparsely settled mountainous region just east of the Cumberland Plateau, the very location of which presented serious problems of access from the start. In the case of General Joe Wheeler Dam, on the other hand, so little preliminary work had to be done that construction of the dam itself could begin simultaneously with the launching of the project.

General Joe Wheeler Dam is located at the head of Lake Wilson and about 15½ miles upstream from Wilson Dam. Here the Tennessee River is more than a mile wide and from 6 to 10 ft. deep. A few hundred yards upstream from the dam are the Goose Shoals, around which part of the old Muscle Shoals canal is still in use. (For a map of the Tennessee River showing the location of these dams see *ENR*, May 11, 1933, p. 580.)

The abutments on both sides of the river are abrupt cliffs, ranging from 85 to 100 ft. in height. The terrain extending from the abutments is gently rolling, partly wooded but mostly in cultivation. Main highways parallel the river on both sides, with roadways leading down to the river.

For an agricultural section, the area is well populated, furnishing an adequate supply of labor from near-by farms and villages. Easy access to the dam site makes it possible for many workers to drive to work in their own cars. Others use the bus transportation which the Tennessee Valley Authority provides. While a construction camp is being built, it is contemplated that at no time

will it be necessary to house more than a small part of the men employed.

Also, the Corps of Engineers, U. S. Army, had made exhaustive studies of the dam and flowage areas extending over a number of years. A wealth of data was thereby immediately available.

At Wilson Dam there is a railroad spur and barge landing. Supplies and equipment can be delivered at that point and then transported by water to the General Joe Wheeler Dam site. Adequate river transport was at hand for this purpose. With these advantages, it was comparatively easy to start construction work quickly and maintain a rapid rate of progress.

In October of last year President Roosevelt requested the Tennessee Valley Authority to start work on the dam. Actual construction began about a month later.

General Joe Wheeler Dam will be about 6,400 ft. in length. It will raise the present water level 50 ft. and will form a lake 80 miles long, extending to a point about 5 miles upstream from Guntersville, Ala. The dam will contain about 600,000 cu.yd. of concrete and necessitate an excavation of 500,000 cu.yd. of earth and rock, 300,000 of which will come out of the area inclosed by the first cofferdam. The extreme height of the dam will be about 70 ft.

Included in the dam is a single navigation lock of a lift of 50 ft. and a power house for eight 35,000-hp. hydraulic turbines, a total of 280,000 hp. Construction of the navigation lock was begun in January, 1933, under the direction of the Corps of Engineers (*ENR*, Aug. 17, 1933, p. 196). Exclusive of the lock and power house, the dam will cost $20,000,000. About two years will be required for its construction.

Construction operations

Because work on the navigation lock already was under way at the north abutment when construction of the dam was undertaken, it was decided to concentrate the initial activities on the dam at the south abutment, utilizing the railhead facilities at Wilson Dam. Consequently arrangements were made with the army engineers for the use of their river crafts, both for transport and construction. Included in the equipment thus obtained were the following: two steel derrick boats, one with a boom 86 ft. long and the other with a 67-ft. boom; two wooden derrick boats with booms less than 50 ft. in length; a dipper-dredge boat with 1½-yd. dipper; a split-paddle river power boat, a smaller towboat of the pusher type and a fleet of barges.

Work started Nov. 20, 1933, with the framing of timber for the cofferdam. This work was done at the Wilson Dam reservation. Compressed air from the Wilson Dam machine shops was used for boring the holes in the timbers, for threading bolts and for making drift pins. The next day the clearing of the dam site began, and the first sections of the cofferdam were placed in the river.

The cofferdam incloses the area to be excavated for the power house and the first section of the dam. As the work progresses, it will be extended farther into the river. Its upstream side is 1,400 ft. long, the end wall 400 ft., and the downstream wall 1,450 ft., the difference between the upstream and downstream sides being due to the curvature of the south abutment.

cove just west of the axis of the dam was selected and put into immediate use. Subsequently the cove was improved by the building of a pier and breakwaters, forming a harbor about 200 ft. long and 200 ft. wide with a narrow opening. The breakwaters, made of cribbing filled with stone, are necessary as Lake Wilson becomes rough and choppy on windy days.

Among the buildings erected are a machine shop, drill-bit-sharpening shop, carpenter shop, warehouse, garage, office building, first-aid station, and hous-

THE ELECTRIC SHOVELS used in the foundation work were assembled at Wilson Dam and floated on barges to Joe Wheeler Dam. As here shown, one shovel is being brought to a crib within the cofferdam from which it descended to the river bottom on a ramp after the cofferdam had been unwatered.

The cofferdam is 20 ft. thick, and the height of the sheathing is 16 ft. Height of wales varies from 6 to 10 ft. Pine timbers, 8x8 in., were used in the three lower rows of wales, and 6x8 in. in the upper rows. The filling material is a type of "gumbo," a gravelly muck, taken from a low flat island in the river. This gumbo becomes very tight and hard when dry.

This type of cofferdam is especially adapted to a low head of water where the river or lake bottom is firm and comparatively level. In considering its application to this site it should be borne in mind that there is comparatively little current in the river. The water flows over flat ledges of exceptionally hard and durable limestone. The depth of the river, from 6 to 10 ft., is fairly constant, and the water level, even in flood times, varies only a few feet because Wilson Dam is so constructed that it can discharge enormous quantities of floodwater, thus maintaining Lake Wilson at its normal level at all times. The cofferdam accordingly is built to withstand a maximum head of 15 ft.

Work at the dam began on Nov. 21. Among the first requirements was a barge landing. For this purpose a little

ing for the air compressors and an electrical substation.

To supply power to the dam site, a 17-mile transmission line had to be built from Wilson Dam. This line was completed in exactly two weeks from the setting of the first pole. It provides a 12,000-volt service of 3-phase 60-cycle alternating current. At the substation this is stepped down to 2,300 volts for distribution to various parts of the construction area, with additional transformers placed where needed to reduce the voltage still further. All mechanical equipment, including the power shovels used in excavation work, is electrically driven.

Compressed air for drilling operations is supplied by two compressors, each having a capacity of 2,100 cu.ft. of free air per minute, and each driven by a 400-hp. electric motor. Most of the time only one compressor is operated and the other kept in a stand-by condition. Air is conveyed to the cofferdam area by an 8-in. pipe line.

For use in excavating the cofferdam area, the Tennessee Valley Authority purchased two all-electric, full-revolving power shovels, each with a 2½-yd. dipper and mounted on crawler

tractors. These shovels were received at the Wilson Dam railhead in a knock-down condition and assembled in the shops at Wilson Dam. Weighing more than 100 tons each, it was impracticable to convey them to the site by road. Accordingly, they were loaded upon barges and towed up the river.

The first shovel arrived before the cofferdam was completed. A rock-filled crib had been constructed inside the cofferdam area, upon which this shovel was to be placed until such time as the area was unwatered. On the day of delivery to the dam site, the barge containing this shovel was pushed inside the area and moored at the crib. Then, with the aid of hawsers from a towboat, the shovel was pulled off the barge and set upon the crib.

The second shovel arrived after the cofferdam had been completed but before the unwatering had begun. To get this shovel to a crib inside the cofferdam, it was landed on a crib constructed on the outside of the cofferdam, and steel girders placed from one crib to the other were used to form a platform over which the shovel could proceed under its own power. This entire operation, as in the case of the delivery of the first shovel, was carried out on schedule time without interruption or mishap.

The cofferdam was completed and the area unwatered on Jan. 21, exactly two months from the time the first brushwood was cut in the clearing of the dam site. It is estimated that the cofferdam area contained 34,000,000 gal. of water. This was pumped out in less than 72 hours through the use of three 12-in. and two 6-in. centrifugal pumps, electrically driven.

The area to be excavated for the power house and tailrace is 800 ft. long and 200 ft. wide, wedge-shaped, with a maximum depth of 50 ft. Out of this area 300,000 cu.yd. of rock are to be taken.

In the drilling nine wagon drills and twelve jackhammers are being used. Forty per cent dynamite is used in the blasting.

Blasted rock is taken out of the cofferdam area by six 10-ton auto trucks. Part of this rock is being used to form a fill in the riverbed west of the cofferdam, over which a roadway will be constructed connecting with a road now under construction on the south bank. The rest of the rock will be used in extending the cofferdam.

For pouring of concrete in the dam, four portable concrete-mixing plants, mounted upon barges, have been purchased. Each unit will be equipped with a 75-ft. crane for handling sand and stone and 150-ton bins for the storage of materials. Other equipment includes six revolving cranes, each with a 95-ft. boom, to be used for transferring concrete from the mixers to the forms.

Cement for the dam will be brought up the river by barges and pumped into the portable mixing plants.

The construction camp, located a short distance from the construction area, will house about 600 workers. One bunkhouse for 120 men has been completed, and the cafeteria mess hall is in operation. Other units are in the course of construction.

In addition, fifteen permanent residences for the engineering personnel and for use of employes who will later operate the dam and power house are under construction.

All of this housing is being erected by the division of land planning and housing of the Tennessee Valley Authority.

Construction of the dam is under the direction of Arthur E. Morgan, chairman of the Tennessee Valley Authority. The designer is John L. Savage, chief designing engineer, U. S. Bureau of Reclamation. C. H. Locher is consultant on construction. The writer is construction engineer.

Lee H. Huntley is construction superintendent, G. K. Leonard assistant construction engineer, and B. S. Philbrick assistant construction superintendent.

Sewage-Plant Survey—III

Data From Schenectady

THE FOLLOWING ARTICLE is the third of a series presenting in brief form the salient design and performance characteristics of American sewage plants. Articles already published are: Decatur, Ill., March 29, 1934, p. 409; Cleveland, Ohio, April 12, 1934, p. 480.

DURING 1933 the treatment units at the Schenectady, N. Y., sewage plant reduced by 84 per cent the B.O.D. of that part of the city's sewage subjected to purification. The plant comprises a coarse screen, Imhoff tanks, trickling filters and sludge beds. The effluent from the plant is discharged into the Mohawk River. Principal design characteristics and the average 1933 performance figures are shown in the accompanying tabulation.

In Schenectady the sewage is collected by a separate system, but much ground and stormwater enters the sewers and is brought to the treatment plant. As can be seen from the tabulated figures, treatment is provided for less than 65 per cent of the city's sewage. The plant was designed by James C. Harding in 1913 and put in service in 1915, to care for a population of 50,000. In 1920 additional sludge-drying beds were provided, as well as certain plant changes for some additional capacity. These changes were recommended by Metcalf & Eddy, consulting engineers. The total cost of the plant to date is about $300,000.

Volumes of flow given below were determined by counters in the dosing tanks. Analytical results refer to composite samples.

The figures have been made available by Morris M. Cohn, sanitary engineer, in charge of the plant.

SEWAGE-PLANT PERFORMANCE DATA, SCHENECTADY, N. Y.

GENERAL DATA

Population, 1933	93,000
Population served 1933 (holiday equiv.)	60,000
Rated capacity of plant, m.g.d.	7.5
Sewage flow	
Average m.g.d.	8.9
Maximum day, m.g.d.	10.5

SEWAGE-TREATMENT UNITS

Coarse Screens	
Number of units	1
Width of opening, in.	1½
Submerged area of screen, vertical projection, sq.ft.	21
Angle of screen with horizontal, deg.	60
Method of cleaning	manual
Imhoff Tanks	
Number of units	9
Plan of dimensions, each unit, ft	94x29
Settling compartment	
Surface area, each unit, sq.ft.	2,320
Depth to slot, ft.	7
Area of cross-section, sq.ft.	132
Sludge compartment, each unit, cu.ft.	8,200
Gas-vent area, each unit, sq.ft.	343
Sewage flow, all units, m.g.d.	8.9
Detention period, all units (actual flow), hours	2.3
Frequency of sludge removal, each unit, times per year	15
Trickling Filters	
Number of units	6
Plan dimensions, each unit, ft.	30x76
Sewage flow, all units, m.g.d.	8.9
Type of filter material	crushed limestone
Depth of filter material, ft.	5
Type of distributing system	spray nozzle
Type of underdrain system	half-tile

SLUDGE DISPOSAL

Sludge-Drying Beds	
Number of units	10
Total area, acres	0.87
Depth of sand, in.	15
Effective size of sand, mm.	0.25
Depth of sludge applied, in.	10

ANALYTICAL RESULTS

	Settleable Solids, p.p.m.	Suspended Solids, p.p.m.	Total Solids, p.p.m.	Five-Day B.O.D. p.p.m.
Raw sewage	141	172	556	119
Imhoff-tank effluent	...	46	431	82
Trickling-filter effluent	...	31	...	19

SCREENINGS, GRIT AND SLUDGE

	Volume, Cu.Ft. Per M.G.	Dry Solids Per Cent	Volatile Solids Per Cent	pH	Disposal
Coarse screenings	1.4	...	...	...	Burial.
Imhoff-tank sludge	100	5.4	65.2	6.8	To drying beds
Dried sludge	26	35	65.2	...	To farmers.

Vibrated Concrete Pavements—I

Summary of experiments in Missouri and in Michigan and by the Bureau of Public Roads in consolidating pavement concrete by vibration—First of two groups of articles on the most important research of recent years in pavement construction processes

TO OBTAIN the advantages of dry mixtures, of 1 in. or less slump, and not lose the workability which favors uniformity and solidity and also permits present speeds in paving, concrete roadbuilders have in the last two years given serious study to compaction by vibration. Together the results of these studies presage a new art in placing concrete pavement and new high standards in the strength and density of pavement concrete. The work has not yet brought full coordination of results and unity of conclusions, but a long step has been taken in amassing the basic information. For the first time this information is summarized comprehensively by research engineers in the following articles.

These articles are, for publication reasons, presented in two groups, the first of which is in this issue. To visualize clearly the character of the investigations made, one of them, that in Missouri, is described in some detail; in case of the others the results are summarized. One article is a review of experience and observation in the state, New Jersey, that has longest used vibration; another is a discussion of plant requirements for construction by vibration.

As an example of activity in grasping possibilities of improved construction methods and of putting them to test, these studies of vibration are in line with a large body of research carried on recently on many elements of highway design and construction. Fundamentally, the tests described establish: (1) the economic and mechanical practicability of compacting concrete in pavement by vibration; (2) a definite gain in strength, density and economy of materials. They reveal also that vibration is a construction process calling for coordinated plant and procedure, not merely the introduction of an additional unit of roadbuilding equipment.

Missouri Conducts Extended Test

Vibration found to reduce costs without sacrifice of quality, but desirability of machine spreading is indicated

By F. V. Reagel
Engineer of Materials, State Highway Department, Jefferson City, Mo.

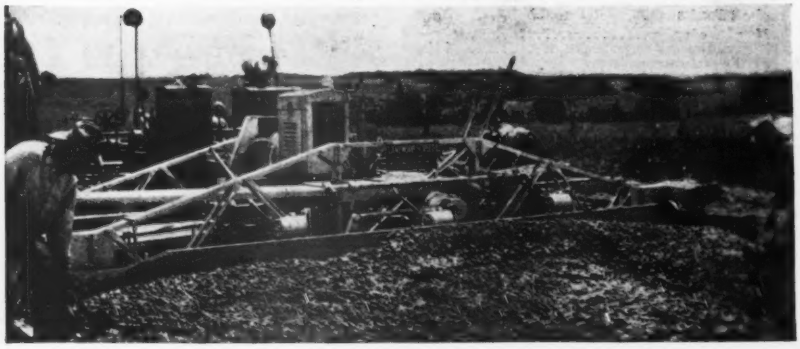
THE MISSOURI state highway department conducted a limited investigation of the use of vibration in the placement and finishing of pavement concrete as a part of a federal-aid project on U. S. Route 50 in Moniteau County. The pavement was of A.A.S.H.O. standard design, a 9:7:9-in. slab 20 ft. wide, with integral lip curbs and expansion joints every 40 ft. and 45 lb. per square of bar-mat reinforcing.

Principally, the investigation consisted of observations on placement, finishing and core strength of (1) concrete of standard proportions, finished in the ordinary manner, with standard equipment, and (2) concrete of leaner proportions and drier consistencies, spread by hand but puddled and finished with a special vibratory finishing machine.

For standard construction a 1:2:3.58 mix of about a 2-in. slump was spread by hand, puddled and finished with a standard finishing machine equipped with an auxiliary tamping bar, and finally floated transversely with wood handfloats.

For vibration, mixtures leaner or drier than the standard mixture and having ½- to 1-in. slump, were handled and finished as under the standard procedure, except that a special vibratory finishing machine was used. This machine was similar in design and operation to a standard finishing machine, except that the screeds were wider and heavier, and the front screed was equipped with three and the rear screed with two electric-motor vibrators (Figs. 1 and 2).

FIG. 1—VIBRATORY FINISHING MACHINE. The three horizontal cylinders attached to the screed are the electric-motor vibrators. Note the rocky texture and dryness of the concrete.



The aggregates, crushed limestone and rounded-grain river sand, were proportioned on the basis of dry-rodded volumes, but were batched by weighing. They analyzed as follows:

	Sand	Stone
Spec. grav. (apparent)	2.64	2.65
Wt., cu.ft., dry-rodded	112 lb.	98 lb.
Voids, dry-rodded, %	32	41
% passing 1½ in. (sq. mesh)	.....	94
" " 1 " " "	.....	60
" " ¾ " " "	100	39
" " ½ " " "	99	7
" " ⅜ " " "	.....	1
" " No. 20	50	.....
" " No. 48	10	.....

Preliminary experiments

Preliminary experiments were made on several mixtures, using the vibratory finishing machine to obtain some idea of the range in consistency and mix that could be handled and placed. These mixtures are shown in Table I.

Observations at once indicated that the vibratory finishing machine was very effective for puddling and finishing dry mixtures—that is, mixtures having a slump of about ½ in. In fact, it was ascertained that concrete of dry consistency and relatively low mortar

TABLE I — PRELIMINARY MIXTURES TESTED TO ARRIVE AT REGULAR TEST MIXTURES

Mixture	Proportions	Cement Factor	Slump	Water Gal per Sack
1	1:2:3.58	1.44	2 in.	5.33
2	1:2:3.58	1.46	½	4.75
3	1:1.8:3.7	1.45	½	4.83
4	1:1.6:4.0	1.47	½	4.79
5	1:2:4.5	1.30	½	5.26
6	1:2.5:5.0	1.11	½	6.00

could be reduced, and (2) to compare the strength and quality of relatively lean vibrated concrete with the standard non-vibrated concrete. This series also consisted of four mixtures, including No. 3 (cement factor 1.44 bbl. per cu.yd.) of the first series. The other three mixtures were designed for lower

of additional hand labor for spreading the concrete, and made difficult proper molding of the lip curb and installation of the mastic in the center groove.

Some batches of the mixtures containing the lower percentages of sand segregated as they were discharged onto the subgrade, the coarse aggregate con-

TABLE II — PROPERTIES OF MIXTURES

Description	Standard Mix	Series I Mixtures No. 1	No. 2	No. 3*	No. 4	Series II Mixtures No. 5	No. 6	No. 7
Proportions by dry rodded volumes	1:2:3.58	1:1.85:3.70	1:1.77:3.91	1:1.68:4.08	1:1.58:4.25	1:2:4.50	1:2.2:4.85	1:2.5:5.0
Cement, bbl. per cu.yd., aver.	1.44	1.47	1.45	1.44	1.42	1.30	1.20	1.13
Water, gal. per cu.yd., actual, aver.	31.2	28.7	28.2	28.0	28.2	27.1	27.5	27.2
Water, gal. per sack of cement aver.	5.44	4.90	4.87	4.88	4.95	5.21	5.72	6.00
Sand, % total aggregate (solid volume)	39.0	36.5	34.0	32.0	30.0	33.7	34.1	36.4
Stone, % total aggregate (solid volume)	61.0	63.5	66.0	68.0	70.0	66.3	65.9	63.6
Yield, cu. ft. of concrete per sack of cement	4.70	4.60	4.66	4.70	4.74	5.21	5.62	5.95
Concrete, % stone by solid volume (b)	45.5	47.9	49.8	51.5	53.2	51.3	51.2	49.8
Workability factor b/b_o	76.6	80.6	83.9	86.7	89.5	84.6	84.5	83.9
Excess mortar, per cent†	75	59	48	38	29	39	39	48
Number of test sections	7	2	1	4	1	4	3	1
Strength, aver. of cores from all test sections	4,490	5,050	5,450	5,020	5,075	4,920	4,290	4,280

*Also a Series II mixture.
Series I mixtures were each designed for a cement factor equal to that of the standard mixture, for a slump of ½ in. and vibration finishing.
Series II represents mixtures in which the cement factor was arbitrarily decreased. These were also designed for ½-in. slump, and vibration finishing.
The value of b_o, the solid volume of stone per unit volume dry-rodded, was the same for all mixtures, 59.4%.
†Per cent excess mortar equals total volume of mortar divided by the void space in the coarse aggregate, minus 1.

content was essential to satisfactory operation. When used with the standard mixture (No. 1, Table I), the vibrators worked a large quantity of mortar to the surface, which could not be incorporated in the pavement and had to be wasted over the side forms before final floating and straightedging. A reduction in mixing water of 0.58 gal. per sack (Mix No. 2, Table I) did not prevent bringing an excess of mortar to the surface, even though the slump was reduced from 2 in. to about ½ in. Mixture No. 3, in which the excess mortar was reduced from 75 to 59 per cent, was handled by the vibrators without waste of mortar but apparently contained more mortar than necessary. Mixtures 4, 5 and 6 were handled satisfactorily.

Test mixtures adopted

Guided by these preliminary experiments, two series of mixtures were designed for finishing with the vibratory finishing machine. In the first series of four mixtures (Table II), the cement factor was kept about constant and equal to that of the standard mix, and the per cent of sand in the total aggregate was varied. The purpose of this series was:

1. To determine what improvement in strength and quality of concrete could be obtained by using mixtures of constant cement content but of lower consistency and mortar content (mixtures approximately equal in cost).

2. To determine the best ratio of sand to total aggregate (with the materials and cement content used) for vibratory finishing.

The second series of mixtures (mixtures 3, 5, 6 and 7, Table II) was designed (1) to determine the practical limit to which the cement content of mixtures finished with the vibrator

TABLE III — EFFECT OF VIBRATORY FINISHING ON COST OF MATERIALS

Mixture	1:2:3.58	1:2:4.5	1:2.2:4.85	1:2.5:5
Cement at $1.96 per bbl.	$2.82	$2.549	$2.352	$2.215
Sand at $1.31 per ton	.845	.759	.772	.827
Coarse agg. at $2.20 per ton	2.23	2.522	2.520	2.450
Total cost per cu.yd.	$5.895	$5.830	$5.644	$5.492
Total cost per sq.yd.	1.28	1.268	1.226	1.193
Total cost per mile	15,050	14,870	14,390	14,010
Saving per mile over standard	.....	$180	$660	$1,040

cement factors, and somewhat higher sand contents were set arbitrarily with the idea of compensating for loss of workability in the mortar due to reduction of the cement.

General observations

The scope of the experiments was limited by difficulties in attempting to adapt ordinary equipment and construction methods, not connected with the operation of the vibratory finisher, to the handling of concrete that was unworkable according to ordinary standards. The dry consistency and the lower cement content made necessary the use

FIG. 2—CLOSE-UP of an electric-motor vibrator, showing method of attachment to rear screed.

centrating along the edges of the batch. At the junction of two such batches quite a pocket of coarse aggregate formed. Cores subsequently drilled from one of these spots were somewhat honeycombed (Fig. 3), showing that vibratory finishing cannot be expected to offset the lack of uniform spreading of the concrete. However, the amount of honeycomb in these cores was much less than the investigators expected to find when the area of segregation was first noted.

Observations of the use of the Series I mixtures indicated that for the cement factor 1.45, and the materials and the concrete-spreading methods used, 32 per cent was the lower safe limit for sand content. Even at this per cent of sand, it was difficult to spread the concrete by hand, and there was considerable segregation of the coarse aggregate. With the 30 per cent sand mixture, the difficulty of hand spreading was extreme, and the degree of segregation was considered by the observers to be so great that the pavement would be badly honeycombed. Cores drilled later did not verify this. In the second series of tests the 1:2.5:5 mixture (No. 7, Table II) was considered to be the end point in reduction of cement factor, for the reasons set forth above.

The hand-spreading methods, with the consequent segregation, probably prevented full development of the ability of the vibratory finisher to handle dry harsh mixtures; it was the observers'

FIG. 3—CORES DRILLED from a coarse-aggregate pocket caused by segregation due to non-uniform spreading of the concrete.

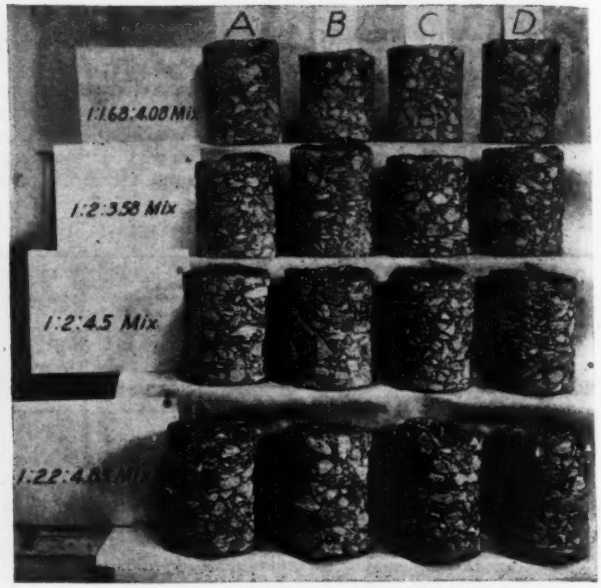


FIG. 4—CORES from increasingly leaner mixtures. Note the difference in the coarse aggregate concentration.

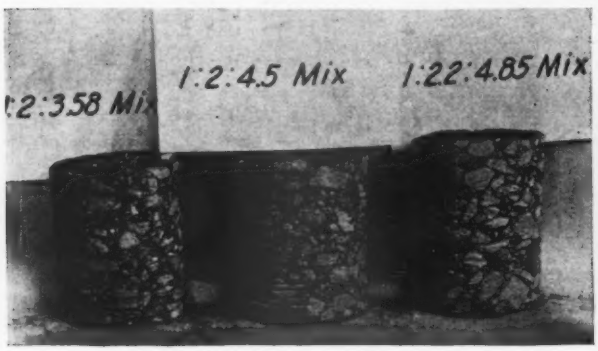


FIG. 5—TYPICAL CORES from several of the mixtures show homogeneous distribution of aggregate.

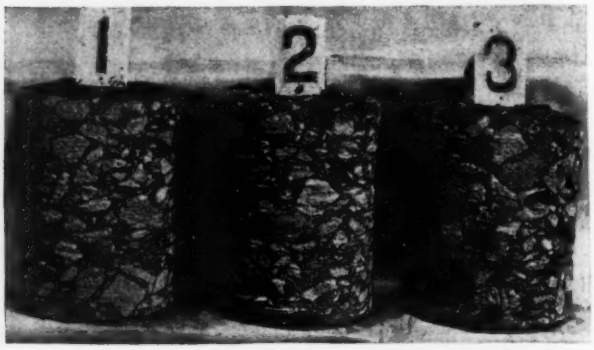


FIG. 6—THESE CORES are typical of the three classes used in grading all the cores visually.

opinion that proper mechanical spreading would have permitted the use of mixtures of still lower sand, cement and water contents. The use of a mechanical spreader, pouring an occasional wet batch for the lip curb, and the use of a steel center joint should eliminate the difficulties encountered.

The surface of the pavement finished with the vibratory finisher was excellent. The concrete was harder and seemed to set quicker than the unvibrated concrete; also there was a thinner layer of mortar on the surface, and practically no free water, which made it possible to cover the pavement with wet burlap almost immediately after finishing.

Observation of the various test sections on the morning following the day of placement, and at later dates up to one year, showed no differences in surface characteristics, all the sections being in excellent condition. Examination of the edges of the pavement below the lip curb showed a little honeycombing, but no more in test sections of one mix and finishing method than in other sections of different mix and finishing method. However, the edges of the lip curb (placed and finished by hand) on the sections in which dry lean mixes had been used were badly honeycombed, much more than the sections in which the standard mix had been used.

Test cores

Visual examination and rating (for porosity and honeycomb) of cores drilled from these test sections indicated that the vibratory finisher had handled all the mixtures of Series I equally well, even the 30 per cent sand mixture. It is, of course, possible that for some of the mixtures that were used on only one test section the number of cores drilled (five from each test section) was insufficient to indicate the true condition of the pavement. For Series II mixtures the cores from the one section poured with the lowest cement content (1.14 bbl. per cu.yd.) were somewhat inferior to the standard sections, indicating that with the materials and methods used no further reduction could be made in the cement content without injuring the quality of the concrete. Cores from the other mixtures were as good as the standard or better.

Strength tests

Five cores were drilled from each test section when 23 days old and tested wet when 28 days old, being treated exactly alike in the interim. In considering the test results and the remarks below, it should be remembered that these are tests of cores drilled from full-scale field construction, and the results are not as accurate as would be obtained under laboratory conditions. Even where quantitative values are given, they are considered to be only qualitative.

All the Series I mixtures, which had the same cement content and hence the same yield as the standard, gave higher strengths than the standard. The difference, based on the average for the four mixtures, was 660 lb. per sq.in., or a 14.7 per cent increase over the stand-

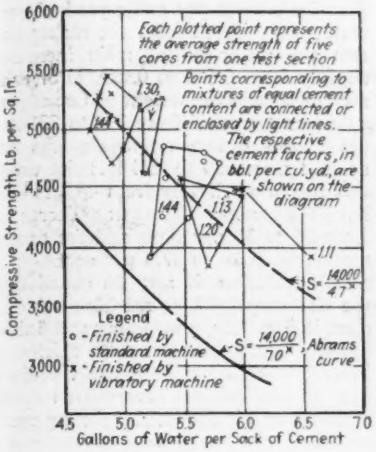

Each plotted point represents the average strength of five cores from one test section
Points corresponding to mixtures of equal cement content are connected or enclosed by light lines.
The respective cement factors, in bbl. per cu. yd., are shown on the diagram
Legend
o - Finished by standard machine
x - Finished by vibratory machine
S = 14,000 / 7.0^x
Abrams curve
S = 14,000 / 4.7^x
Compressive Strength, Lb. per Sq. In.
Gallons of Water per Sack of Cement

FIG. 7—RELATION of strength of cores to water content of the concrete.

ard. Three of the mixtures of Series I had practically identical strengths, and the other was about 400 lb. per sq.in. higher. However, as there was only one test section for each of these mixtures, the result was not considered particularly indicative of a higher strength for this particular mixture. The fact that all four mixtures had practically the same water-cement ratio lends weight to this idea.

A peculiarity of the Series I mixtures was that despite a range of 6.5 per cent in the sand content the water requirement to produce equivalent workability was practically the same for all mixtures. The workability was judged principally on the ability of hand labor to spread and puddle the concrete. It is very possible that with mechanical spreading the water content could have been reduced as the sand content was lowered without affecting the workability but resulting in an increase in strength. This would be in line with theory and also the laboratory results of other investigators.

With the exception of the leanest mixture (No. 7, Table II), all the mixtures of Series II were used on three or four test sections. The mixture with 1.30 bbl. of cement per cubic yard, 0.14 bbl., or 9.7 per cent less than the standard, had an average strength of 430 lb. per sq.in., or 9.6 per cent more than the standard. The mixture with a cement factor of 1.20, a reduction of 16.7 per cent, had an average strength of 200 lb. per sq.in., or 4.5 per cent less than the standard. The leanest mixture with a cement factor of 1.13, a reduction of 0.32 bbl. per cubic yard, or 22 per cent, had a strength about equivalent to the mixture with 1.20 cement factor. This mixture was used on only one test section, so the result is not conclusive. However, none of the cores drilled from it tested lower than 3,900 lb. per sq.in., indicating the mixture to be satisfactory in so far as strength is concerned. These strength tests indicate that, with properly designed mixtures and vibratory finishing, considerably stronger concrete can be obtained with no increase in cost, and considerably less expensive concrete can be used without sacrifice of strength.

Regardless of proportions of materials or method of finishing, the average strengths of the various test sections plotted against the water-cement ratio delineated a curve of characteristic trend (Fig. 7). The implication is that the strength of vibrated, as well as unvibrated concrete, is governed by the water-cement ratio law, vibration merely increasing the range of mixtures that may be considered workable.

The individual core tests indicated that pavement finished with the vibrator was somewhat more uniform in strength than that on which the standard finishing method was used.

Effect on cost

The cost of the materials for several of the mixtures is given in Table III. It is evident from this table that a saving in cost of material can be effected by use of the lean mixtures that can be placed with the vibratory finisher. The 1:2.2:4.85 mixture, which, according to all the tests was as good as the standard, cost $660 per mile less, and the 1:2.5:5 mixture cost $1,040 per mile less than the standard. As previously pointed out, proper spreading equipment used in conjunction with the vibratory finisher might permit the use of still leaner mixtures with even greater reduction in cost of materials.

Conclusion

The results of this experiment show the vibratory finisher to be effective in handling dry lean mixtures and definitely indicate the possibility of reducing the cost of pavement materials without sacrifice of quality. However, further investigation is needed to determine the durability of these lean mixtures and to develop optimum mixtures for use with this type of finisher.

Beam Test Results in Michigan

Difficulties of hand spreading and finishing decreased the utility of vibration, but the concrete proved to be stronger

By John W. Kushing
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TO EXPLORE the possibilities of economy and improved quality in concrete-pavement construction by the use of vibratory finishing machines, the Michigan state highway department conducted a comparative investigation on vibratory and regular finished sections of pavement. The investigation proper was conducted in 1933, but preliminary tests were made in 1932. The preliminary tests demonstrated:

1. That vibratory machines could handle extremely dry concrete.
2. That 1:2:2½ mix by volume contained excess mortar for this type of finishing.
3. That mixes having ½-in. slump or less were difficult to spread in front of the finishing machine by hand, and that it was practically impossible for hand finishers to remove high spots.
4. That it was impossible to place premolded joint material to proper depth and alignment with such mixes.

In view of the above conclusions, it was decided to use in the investigation proper only such mixture as could be spread and finished in the routine manner with results comparable to standard finishing methods.

Testing methods

The investigation was conducted on Trunk Line Highway No. 78, near Lansing, Mich., where a 9-in. uniform pavement 20 ft. wide was being constructed. Reinforcement was used in the greater part of its length. The subgrade varied from clay and clay loam to sand.

Gravel and crushed limestone were used as coarse aggregate, but as gravel was being used throughout the project, greater attention was given to this material. Coarse aggregates were furnished in two sizes proportioned 50 per cent of each by weight and batched by the same method. The mixtures were designed on the mortar-voids theory.

Vibration was secured by means of electric-motor vibrators and was transmitted to the concrete through screeds on the finishing machine, the front screed being equipped with three vibrators and the rear with two.

The investigation can be considered as divided into twelve sections according to the different mixes used as shown in Table IV. Design data are also shown in this table for the various mixes, and the results as to modulus of rupture are shown in Table V. The

TABLE IV — DESIGN OF CONCRETE MIXTURES FOR VIBRATION TESTS OF CONCRETE PAVEMENT BY MICHIGAN HIGHWAY DEPARTMENT

Section	R.W.C.	Excess Paste Per Cent	b/b_o	a_m/a_o	w/c	N Sacks	N_m Sacks	Mix per sack of cement: Sand, Lb.	C.A., Lb.	Water, Lb.	Volumetric Proportions (Approx.)	Specific Gravity	Voids, Per Cent
Gravel													
*1-G-S	1.23	20	.76	.80	.692	6.00	11.10	203	350	43.2	1:1.92:3.43	Cement, 3.12	
1-G	1.35	16	.85	.84	.684	5.10	10.41	231	460	42.7	1:2.18:4.51	Sand, 2.66	
2-G	1.35	16	.80	.84	.684	5.40	10.41	231	409	42.7	1:2.18:4.01	Gravel, 2.70	
3-G	1.35	20	.85	.80	.638	5.64	11.53	199	415	39.8	1:1.88:4.07		36.1
4-G	1.35	12	.80	.88	.758	4.82	9.29	271	457	47.3	1:2.56:4.48		
5-G	1.35	15	.875	.85	.699	4.80	10.13	240	502	43.6	1:2.26:4.92		Sand, 39.5
6-G	1.40	18	.825	.82	.699	5.36	10.64	221	424	43.6	1:2.08:4.16		Gravel,
Stone													
*1-S-S	1.23	20	.76	.80	.729	6.00	10.20	232	308	45.5	1:2.19:3.42	Cement 3.12	
1-S	1.35	16	.825	.84	.684	5.77	10.41	231	347	42.7	1:2.18:3.86	Sand, 2.66	
2-S	1.35	16	.85	.84	.684	5.63	10.41	231	367	42.7	1:2.18:4.08	Stone, 2.67	
3-S	1.30	16	.875	.84	.638	5.70	10.80	223	374	39.8	1:2.10:4.16		Sand, 36.1
4-S	1.35	16	.85	.84	.684	5.63	10.41	231	367	42.7	1:2.18:4.08		Stone, 46.0

Nomenclature and explanation: *Standard mix not vibrated.
R.W.C. — relative water content.
Excess paste — per cent by volume.
b/b_o — vol. of loose coarse aggregate in a cu.ft. of concrete.
a_m/a_o — vol. of loose fine aggregate in a cu.ft. of mortar.
w/c — water-cement ratio by volume.
N — sacks of cement per cu.yd. of concrete.
N_m — sacks of cement per cu.yd. of mortar.

concrete beam-test specimens, 6x8x36 in., were molded from concrete incorporated in the work, and every effort was made to produce concrete in the test molds like that produced by the vibratory finishing machine in the pavement. The vibration of the molds was obtained by means of a table with vibrator attached.

Conclusion from tests

Any conclusions to be drawn at this time must necessarily be based on field observation and the beam breaks, as the cores that were taken will not be broken until one year old.

Analyzing field reports, it was found that mix 6-G was the most nearly ideal from the standpoint of mixing, placing, handling and finishing. A smooth surface was obtained, and it was possible to install the longitudinal contraction joint in a satisfactory manner, these results being secured with the same amount of labor as with standard gravel mix. Because of these facts about 4,320 lin.-ft. of this mix were laid.

In comparing mix 6-G with the standard gravel mix it will be noted that the 7- and 28-day flexural strengths average practically the same for a total of 14 beam-test specimens, or 28 breaks for each period of time.

It is also to be noted that in general the slumps for the standard mix and 6-G were 2½ and ¾ in., respectively; that the cement per cubic yard of concrete was reduced from six sacks; and that the water-cement ratio increased slightly from 0.692 to 0.699. For each sack of cement the weight of fine aggregate was increased from 203 to 221 lb.; the gravel coarse aggregate was increased from 350 to 424 lb.; the water was increased slightly from 43.2 to 43.6 lb. The excess paste was decreased from 20 to 18 per cent, and the excess mortar from 24 to 17.5 per cent.

In conclusion, it may be stated that the vibratory finishing machine was efficient in handling, compacting and finishing dry harsh mixtures. Its efficiency, however, could not be entirely utilized due to difficulties encountered with hand-spreading and hand-finishing operations.

In spite of this fact, early results of the investigation seem to indicate the possibility of reducing the cost of concrete in pavements, without sacrifice of quality, by reducing the cement content per cubic yard of concrete.

TABLE V — DATA OF MIXTURES OF GRAVEL AND STONE AND RESULTS OF STRENGTH TESTS OF CONCRETE BEAMS MADE DURING INVESTIGATION OF VIBRATORY FINISHING MACHINE

Section	Mix Per Sack of Cement: Sand, Lbs.	Coarse Aggregate: 4A Lb.	10A Lb.	Total Lb.	Water: Theo. Lb.	Theo. Gal.	Used Lb.	Used Gal.	Water-Cement Ratio: Theoretical	Actual	Slump, Inches	Modulus of Rupture Lb. Per Sq.In. 7 days: No. of Breaks	Aver.	28 days: No. of Breaks	Aver.
Gravel															
1-G-S	203	175	175	350	43.2	5.2	44.9	5.4	.692	.719	2½	28	668	28	805
1-G	231	230	230	460	42.7	5.1	49.5	5.9	.684	.793	¾	4	673	4	826
2-G	231	204.5	204.5	409	42.7	5.1	50.9	6.1	.684	.816	¾	4	735	4	810
3-G	199	207.5	207.5	415	39.8	4.8	47.4	5.7	.638	.760	¾	4	716	4	906
4-G	271	228.5	228.5	457	47.3	5.7	52.4	6.3	.758	.840	¾	4	518	4	617
5-G	240	251	251	502	43.6	5.2	51.9	6.2	.699	.832	¾	4	523	4	707
6-G	221	212	212	424	43.6	5.2	46.7	5.6	.699	.749	¾	28	663	28	785
Stone															
1-S-S	232	154	154	308	45.5	5.5	53.0	6.4	.729	.849	2	4	631	4	830
1-S	231	173.5	173.5	347	42.7	5.1	44.2	5.3	.684	.708	½	4	628	4	871
2-S	231	183.5	183.5	367	42.7	5.1	48.0	5.8	.684	.769	½	4	599	4	800
3-S	223	187	187	374	39.8	4.8	48.0	5.8	.638	.769	½	4	783	4	937
4-S	231	183.5	183.5	367	42.7	5.1	50.8	6.1	.684	.814	½	4	633	4	767

Aggregates Compared by B.P.R.

Tests show conservatively a general improvement of paving concrete by vibration, depending on character of aggregate

By F. H. Jackson
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and W. F. Kellermann
Associate Materials Engineer, Bureau of Public Roads, Washington, D. C.

IN 1931 a report was published describing the results of an investigation conducted by the Bureau of Public Roads to determine the effect of certain controllable variables on the quality of pavement concrete. Observations and tests were made on actual pavement sections, placed and finished under working conditions.

In the tests reported, a study was made of the effect of varying the quantity of coarse aggregate in the mix for various types and gradings of coarse aggregate and for various consistencies. The concrete was mixed and placed in accordance with accepted construction practice, which included finishing by the use of the conventional type of power-driven screeds, with and without tamper attachment.

It was definitely determined that with the methods of machine finishing now commonly employed a consistency corresponding to a slump of at least 2 in. must be used if honeycomb is to be avoided. Of course, such a concrete does not have as high a potential strength and density as a somewhat drier mix. Actually, however, it was found that both strength and uniformity

were improved by the use of the wetter consistency. This result is explained by the fact that the segregation and honeycombing that accompany the use of the drier mix actually tend to lower the strength of the pavement slab to a point considerably below the strength that would be expected theoretically from the water-cement-ratio strength law. This means that under our present construction practice concretes mixed in the proportions used in paving work and with normal aggregates are not "workable" as paving mixes if the slumps are less than 2 in.

A realization of this fact led naturally to a consideration of methods of placing which would permit the use of drier mixes without sacrificing strength or uniformity of strength. The use of vibratory screeds for this purpose presented interesting possibilities, and it was decided to continue the experiments, using a standard finishing machine with vibrators mounted directly on the screeds. This article summarizes the results of these tests as reported by the Bureau of Public Roads in their journal *Public Roads*.

Test procedure

A standard double-screed finishing machine was equipped with three high-frequency vibrators, two mounted on the front screed and one on the rear screed. Each vibrator consisted of a ½-hp. electric motor operating at about 3,600 r.p.m. There was attached to the rotor of each motor an eccentric weight which, revolving at high speed, imparted a vibratory motion to the screed. The only modification of the finishing machine as ordinarily used, other than the installation of the vibrators, was the use of a bull-nose strike-off that was riveted to the front screed.

In general, the finishing machine was operated at about the same forward speed as is customarily used. The number of passes was limited to three, this being considered the maximum allowable from the standpoint of production. It was found necessary to exceed two passes in only five cases.

For each combination of fine and coarse aggregate, a so-called base mix was established. This base mix was designed so that the combinations containing the coarse sand would have a cement factor of about six sacks per cubic yard. For the corresponding mixes with fine sand the proportion of sand was reduced so as to compensate for the differences in grading without changing the water-cement ratio or the proportion of coarse aggregate.

Each base mix was designed to produce concrete with a slump of about 2 in. and was used in a section constructed without vibration as a standard of comparison. For each standard section there were constructed five sections in which the concrete was somewhat drier than the standard, all of which were vibrated. In two of the vibrated sections the proportions were the same as the standard or base mix, and the drier consistency was obtained by lowering the water content until slumps of about 1 in. and ½ in. were obtained. In the other three the water-cement ratio used in the standard or base mix was maintained constant, and the drier consistency was secured by adding respectively one-fourth, one-half and three-fourths parts coarse aggregate by volume to the base proportion. A set of six sections, one screeded and five vibrated, formed one day's run.

Summary of conclusions

The major facts developed by this investigation are summarized in the following paragraphs:

1. When finished by the standard method, the minimum slump required to insure pavement slabs substantially free from honeycomb was found to be approximately 2½ in.

2. When finished by vibration, the minimum slump required to insure the same degree of uniformity was found to be approximately 1 in.

3. The average flexural strength of all vibrated pavement slabs in which the concrete showed an average slump of 1 in. was somewhat higher than the average strength of the standard-finished concrete of the same proportions in which the average slump was 2 in. The average increase was due to the marked increase in strength for the group of sections in which crushed stone was used as coarse aggregate. For the sections containing gravel as coarse aggregate, the vibrated concrete having a 1-in. slump showed a lower average flexural strength than the standard-finished concrete having a 2-in. slump. In the case of slag the strengths for the two slumps were about the same.

4. The average flexural strength of all vibrated pavement slabs in which the concrete showed an average slump of ½ in. was lower than the average strength of the standard-finished concrete of the same proportions in which the slump averaged 2 in. The average decrease was due to the marked decrease in strength for the group of sections in which gravel was used as coarse aggregate. In the case of stone and slag the average strength of the vibrated concrete having a ½-in. slump was about the same as the average strength of the standard-finish sections having a 2-in. slump.

5. The average flexural strength of all vibrated pavement slabs constructed of concrete having the same proportions of cement, fine aggregate and water, but containing one-fourth part more coarse aggregate by volume than the standard-finished concrete, was considerably higher than that of the standard-finished concrete. The increase was most marked in the case of the crushed-stone concrete and least in the case of the gravel concrete.

6. The average flexural strength of all vibrated concrete slabs containing one-half part more coarse aggregate than the standard-finished concrete was somewhat higher than that of the standard-finished concrete. The sections containing crushed stone gave considerably higher values, the sections containing slag slightly higher, and the sections containing gravel somewhat lower.

7. The average flexural strength of all vibrated concrete slabs containing three-fourths part more coarse aggregate than the standard-finished concrete was lower than that of the standard-finished concrete. The sections containing crushed stone and slag gave about the same results, whereas the sections containing gravel showed a decrease.

8. For concrete having the same water-cement ratio, finishing by vibration did not increase the crushing strength of cores drilled from the pavement slabs containing gravel and slag as coarse aggregate. In the case of crushed stone a considerable increase in crushing strength was noted for the vibrated concrete containing one-half and three-fourths parts additional coarse aggregate.

9. The process of finishing by vibration did not adversely affect the hardness of the surface.

The second part of this article, to be published next week, will give results of tests and experience in Ohio, Illinois and New Jersey and will discuss equipment for pavement construction by vibration.

Measuring Impact Effect on Concrete Piles

The British Building Research Board, in cooperation with the Federation of Civil Engineering and Public Works Contractors, is carrying out a study of the effect of impact during the drivng of reinforced-concrete piles. The strains set up in the piles during driving are of a momentary character, and an ingenious apparatus has been devised to measure them.

Quartz crystals, such as are used to control the wave length of radio stations, are embedded in the piles being driven. When these crystals are mechanically compressed by the forces set up in the pile, a corresponding electrical force is developed between two metal electrodes deposited on their faces, and this almost instantaneous electrical force is recorded by a cathode ray oscillograph. In a report on the subject just issued by the Building Research Board, illustrations show records made by this new strain gage of stresses amounting to 1,500 to 2,500 lb. per sq.in. developed in the pile as the result of blows lasting 9/1,000th of a second and given by a 980-lb. hammer falling from a height of 1 ft.

Simplified Analysis of Indeterminate Frames

A new procedure based on displacement equations derived from Williot diagrams saves labor in the design of hyperstatic structures—Practical application of method shown by analysis of a four-panel truss and by a bridge-tower problem

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PRESENTING an interesting direct method of analyzing complex framed structures containing redundant members, the author outlines the history and nature of the two methods now in use and describes his new Williot strain method. He first allies the three methods successively to a four-panel truss having double diagonals (four redundant members); then he uses the new method to deal with the intricate case of a structure similar to the George Washington Bridge towers. Finally, he shows how the Williot strain method, when used in conjunction with the method of expressing end moments in terms of angular displacements, furnishes a valuable tool to the structural engineer for dealing with certain types of complicated structures.

THERE ARE two well-known methods for deriving and applying the "elastic" equations necessary for the solution of a statically indeterminate framed structure. One method is known variously as the method of virtual velocities, virtual work, virtual displacements, work, elastic energy, etc. The other is universally known as the method of least work. In this article the two methods will be designated as (1) the method of work and (2) the method of least work.

Methods now used

The principle of work has long been known, but its application to statically indeterminate structures dates from the middle of the last century. Any bibliography in this field would include Lamé, Clapeyron, Ménabréa, Castigliano, Maxwell, Mohr and Winkler, to mention but a few. Lamé and Clapeyron[1] pioneered in this field. Swain[2] and Cain[3] were among the first American writers on this subject. Swain names Lamé[4] as the first writer to apply the method of work to framed structures; he gives the date 1866, which is that of the second edition of Lamé's book, but the same discussion is given in the first edition (1852). Many modern writers seem to recognize Castigliano[5] as the father of the method of least work, although Ménabréa[6] clearly enunciated this method 21 years earlier. We must admit that his proof, based on the hypothesis that no work is done when a force elongates a member a "very small" amount, is not above suspicion; but more than one modern writer has accepted it.

The writer submits that the principle of work and the methods derived therefrom are essentially mathematical abstractions and of a nature that do not permit either physical conception or that space perception which is helpful in the first approach to the comprehension of any principle or method. The writer will describe a new method that he has discovered for solving statically indeterminate framed structures, one which requires no discussion of abstract fundamental principles either for its introduction or its proof, as the proof is self-evident. Perhaps the method is not as new as the writer believes, but an extended search in engineering publication has failed to uncover any reference to it.

This method will first be explained. Then a simple problem will be solved by the three methods for comparison, and later the application of the new method to a more complicated structure will be outlined.

In the method of work a redundant member is assumed to be cut and the gap, say Δ_1, between the two cut ends, caused by strains in all other members, is calculated. Then, if Δ_2 represents the strain (lengthening or shortening) of the cut member necessary to close the gap,

$$\Delta_1 + \Delta_2 = 0$$

Equations 4, 5, 6 and 7, shown later, are of this character; and it will be seen on inspection that each term in these equations represents a strain measured in inches.

In the method of least work the fundamental equation, which is to be differentiated, is an expression for the total internal work performed. The terms are expressed in units of work, and the equation is essentially a "work" equation.

In the new method about to be described, equations are developed in which the terms represent strains, but the derivation is made directly without any detour or reference to the principles of work. The geometry of the Williot diagram provides the simplest and most direct route for deriving these strain equations. For this reason this new method will be designated as the Williot strain method, and the equations will be called the Williot strain equations, to distinguish them from the "strain" or "elastic" equations in the method of work and from the "work" equations in the method of least work.

Deriving the Williot equations

In *Annales du Génie Civil*, October, 1877, there is an article entitled "Notions Pratiques sur La Statique Graphique," signed Williot. This paper deals with ordinary stress diagrams; but in the December number, under the same title and by the same author, two illustrations of deformation diagrams are given and described which have come to be known as Williot diagrams.

In Fig. 1*a*, let *ABDC* represent any panel of a truss having an inclined top chord and two web members; and let Δ represent the strain in any particular member. Assume that all members are in tension, hence all strains are elongations and have positive sign.

Construct the Williot diagram, Fig. 1*b*, assuming one member fixed in direction *BD*, for example. Locate joint *C* by the strains Δ_{bc} and Δ_{cd}, after which the joint *A* may be located either by the strains Δ_{ab} and Δ_{ad} or by the strains Δ_{ab} and Δ_{ac}. From the geometry of the diagram,

$$e = \Delta_{bc} \sec\theta$$
$$f = \Delta_{cd} - \Delta_{bd} \sin\psi$$
$$g = f \tan\theta = (\Delta_{cd} - \Delta_{bd} \sin\psi) \tan\theta$$
$$h = \Delta_{ad} \operatorname{cosec}\phi$$
$$j = \Delta_{ad} \operatorname{cosec}\phi - \Delta_{bd} \sin\psi - \Delta_{ab}$$
$$k = j \tan\phi = (\Delta_{ad} \operatorname{cosec}\phi - \Delta_{bd} \sin\psi - \Delta_{ab}) \tan\phi$$
$$n = \Delta_{bd} \cos\psi$$
$$m = \Delta_{ac}$$

Then $e - g - m = n - k$
or $e + k = m + n + g$

Substituting and transposing,

$$\Delta_{bc} \sec\theta + \Delta_{ad} \sec\phi = \Delta_{ac} + \Delta_{bd}(\sin\psi \tan\phi - \sin\psi \tan\theta + \cos\psi) + \Delta_{ab} \tan\phi + \Delta_{cd} \tan\theta$$

From Fig. 1*a*,

$$\tan\phi - \tan\theta = \tan\psi$$

Whence

$$\Delta_{bc} \sec\theta + \Delta_{ad} \sec\phi = \Delta_{ac} + \Delta_{bd} \sec\psi + \Delta_{ab} \tan\phi + \Delta_{cd} \tan\theta \qquad (1)$$

By making the top chord horizontal, the trapezoid of Fig. 1*a* is transformed into the rectangle of Fig. 2*a*. In this case

$$\psi = 0,\ \sec\psi = 1,\ \phi = \theta$$

and Eq. (1) becomes,

$$\left.\begin{aligned}(\Delta_{bc} + \Delta_{ad}) \sec\theta &= \Delta_{ac} + \Delta_{bd} \\ &\quad + (\Delta_{ab} + \Delta_{cd}) \tan\theta \\ \text{which may be written in the form} \\ (\Delta_{bc} + \Delta_{ad}) \operatorname{cosec}\theta &= (\Delta_{ac} + \\ \Delta_{bd}) \cot\theta + \Delta_{ab} + \Delta_{cd}\end{aligned}\right\} \qquad (2)$$

It may also be demonstrated from a

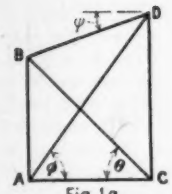

Fig. 1a
Inclined chord panel

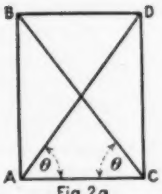

Fig. 2a
Parallel-chord panel

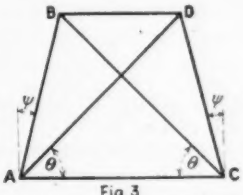

Fig. 3
Viaduct-bent panel

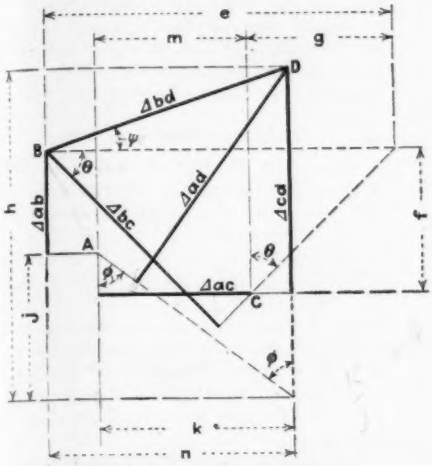

Fig. 1b - Williot diagram for panel Fig. 1a

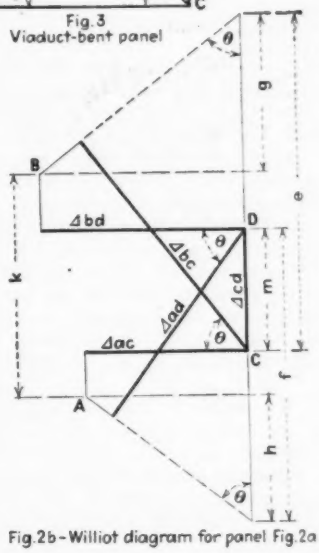

Fig. 2b - Williot diagram for panel Fig. 2a

FIGS. 1-3—THE basic Williot equations are derived from elementary six-bar panels of a frame through the geometry of the Williot diagrams.

Williot diagram drawn for a panel having the pattern of Fig. 3 that,

$$(\Delta_{bc} + \Delta_{ad}) \sec\theta \cos\psi = (\Delta_{bd} + \Delta_{ac}) \cos\psi + (\Delta_{bd} - \Delta_{ac}) \tan\theta \sin\psi + (\Delta_{ab} + \Delta_{cd}) \tan\theta \qquad (3)$$

When $\psi = 0$, the patterns of Fig. 2*a* and Fig. 3 are alike; $\cos\psi = 1$, $\sin\psi = 0$, and Eq. (3) is transformed into Eq. (2).

Eq. (1), (2) and (3) are general Williot strain equations applicable to the three most common types of quadrilaterals found in framed structures. They were derived on the assumption that all strains were elongations corresponding to a tensile stress, but if for any reason it is desirable to assume a compressive stress in any member, say *BC*, it is necessary only to change the sign of Δ_{bc} in the equation. Again, suppose a compressive stress were assumed for all the members, then change the signs of all the terms in the equation, which of course changes the equation not at all; rotate Fig. 1*a* through 180°, and the strains in Fig. 1*b* now correspond to compressive stresses.

Eq. (1), (2) and (3) represent relative and not absolute locations of the displaced joints, consequently *the equations are the same whatever member is chosen as fixed in direction.* To illustrate: the Williot diagram in Fig. 2*b* has been drawn by assuming the member *CD*, instead of *BD*, fixed in position and all strains positive; from which

$$e = \Delta_{bc} \operatorname{cosec}\theta$$
$$f = \Delta_{ad} \operatorname{cosec}\theta$$
$$g = \Delta_{bd} \cot\theta$$
$$h = \Delta_{ac} \cot\theta$$
$$k = \Delta_{ab}$$
$$m = \Delta_{cd}$$

Then $e + f - m - g - h = k$
or $e + f = h + g + k + m$

Whence $(\Delta_{bc} + \Delta_{ad}) \operatorname{cosec}\theta = (\Delta_{bc} + \Delta_{bd}) \cot\theta + \Delta_{ab} + \Delta_{cd}$, which is Eq. (2).

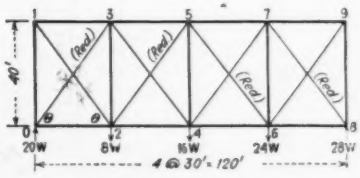

FIG. 4—A SIMPLE CASE of redundancy, with four members more than required in statically determined framing, analyzed by three methods for comparison.

Thus, in any of these panels, whatever member be assumed fixed in direction, a simple equation is obtained by which the strain in the redundant member (which may be any one of the six members of the frame) is found in terms of the strains in the other five members. Through this relation, the stresses can be found, as will now be illustrated.

Comparison of three methods

The truss shown in Fig. 4, having four redundant members, will be used for the application of the three methods. The cross-sectional area A (sq. in.) and the length l (in.) of each member are supposed to be known. The quantities $\frac{l}{A}$ are given in the tables. The modulus of elasticity E is assumed constant for all members.

Method of Work—The proof of this method and the development of the four equations required for the solution may be found in many treatises on the subject and will not be repeated here; the equations will simply be stated with the necessary description of terms. The members 0-3, 2-5, 5-6 and 7-8 will be taken as the redundant members.

Let S' = stress in any member due to loads, as shown, after all redundant members have been removed;

S_{03}, S_{25}, S_{56} and S_{78} = stresses in redundant members (unknown);

u_{03} = stress in any necessary member due to stress of unity in redundant member 0-3 (this member present and all other redundant members removed);

u_{25} = stress in any necessary member due to stress of unity in redundant member 2-5 (this member present and all other redundant members removed); etc.;

The four "elastic" equations derived from the principle that internal work equals external work are as shown in Eq. (4), (5), (6) and (7) below:

$$S_{03}\left(\frac{l_{03}}{EA_{03}} + \sum\frac{u_{03}^2 l}{EA}\right) + S_{25}\sum\frac{u_{03}u_{25}l}{EA} + S_{56}\sum\frac{u_{03}u_{56}l}{EA} + S_{78}\sum\frac{u_{03}u_{78}l}{EA} + \sum\frac{S'u_{03}l}{EA} = 0 \qquad (4)$$

$$S_{03}\sum\frac{u_{25}u_{03}l}{EA} + S_{25}\left(\frac{l_{25}}{EA_{25}} + \sum\frac{u_{25}^2 l}{EA}\right) + S_{56}\sum\frac{u_{25}u_{56}}{EA} + S_{78}\sum\frac{u_{25}u_{78}}{EA} + \sum\frac{S'u_{25}l}{EA} = 0 \qquad (5)$$

$$S_{03}\sum\frac{u_{56}u_{03}l}{EA} + S_{25}\sum\frac{u_{56}u_{25}l}{EA} + S_{56}\left(\frac{l_{56}}{EA_{56}} + \sum\frac{u_{56}^2 l}{EA}\right) + S_{78}\sum\frac{u_{56}u_{78}l}{EA} + \sum\frac{S'u_{56}l}{EA} = 0 \qquad (6)$$

$$S_{03}\sum\frac{u_{78}u_{03}l}{EA} + S_{25}\sum\frac{u_{78}u_{25}l}{EA} + S_{56}\sum\frac{u_{78}u_{56}l}{EA} + S_{78}\left(\frac{l_{78}}{EA_{78}} + \sum\frac{u_{78}^2 l}{EA}\right) + \sum\frac{S'u_{78}l}{EA} = 0 \qquad (7)$$

The various quantities appearing in the equations are computed as shown in Table I, with the summations given at the bottom of each column. Substituting these quantities in Eq. (4), (5), (6) and (7) and omitting the common factor E, we have

$$(50 + 128.72)S_{03} + 30.72S_{25} = -2550.8W \quad (8)$$
$$30.72S_{03} + (60 + 130.72)S_{25} + 25.6S_{56} = -1468.8W \quad (9)$$
$$25.6S_{25} + (100 + 151.76)S_{56} + 15.36S_{78} = -603.8W \quad (10)$$
$$15.36S_{56} + (30 + 79.76)S_{78} = -1952.8W \quad (11)$$

Solution of these simultaneous equations will determine the stresses in the redundant members, after which the stresses in the remaining members may be computed by the principles of statics.

Method of Least Work—Let $5x$, $5y$, $5v$ and $5z$ represent respectively the stresses in the members 0-3, 2-5, 5-6 and 7-8 of the truss just analyzed and assume that all members are in tension. Compute the stresses in the remaining members by the principles of statics, in terms of x, y, v, z and W: these stresses are given under S in Table II. The strain in any member is

$$\Delta = \frac{Sl}{AE}$$

The internal work performed on each member is

$$\tfrac{1}{2}S\Delta = \frac{S^2l}{2AE}$$

and the total internal work is

$$U = \sum \frac{S^2l}{2AE}$$

The quantity $\frac{S^2l}{2A}$ is computed for each member, and the sum for all members is given in the table, whence

$$U = \frac{1}{E}\sum \frac{S^2l}{2A} = \frac{1}{E}(86246W^2 + 12754Wx + 2234x^2 + 7344Wy + 2384y^2 + 3019Wv + 3147v^2 + 9764Wz + 1372z^2 + 768xy + 640yv + 384vz).$$

The partial differentiation of U with respect to x, y, v and z gives the following expressions, which, when equated to zero and solved, give the values of x, y, v and z that will render the value of U a minimum.

$$\frac{\delta U}{\delta x} = 12754W + 4468x + 768y = 0 \quad (12)$$
$$\frac{\delta U}{\delta y} = 7344W + 4768y + 768x + 640v = 0 \quad (13)$$
$$\frac{\delta U}{\delta v} = 3019W + 6294v + 640y + 384z = 0 \quad (14)$$
$$\frac{\delta U}{\delta z} = 9764W + 2744z + 384v = 0 \quad (15)$$

In the previous method the stresses in the redundant members were represented by S_{03}, S_{25}, S_{56} and S_{78}; and in this method by $5x$, $5y$, $5v$ and $5z$. If the latter set of symbols is substituted for the former in Eq. (8), (9), (10) and (11), the equations become identical with Eq. (12), (13), (14) and (15).

Williot Strain Method—Compute the stresses S, as in the previous method. They are shown in Table III. If, in computing the stresses S, it seems more natural to assume that the top chord, verticals and one set of diagonals are in compression, then do so, but make the corresponding changes in signs in Eq. (16) to (19), inclusive. Complete the table by computing $E\Delta = \frac{Sl}{A}$, in which Δ is the strain in any member.

Since the pattern of each panel conforms to Fig. 2*a*, write Eq. (2) for each panel, noting that cosec $\theta = 1.25$ and $\cot\theta = 0.75$:

$$(\Delta_{12} + \Delta_{03})1.25 = (\Delta_{02} + \Delta_{13}) 0.75 + \Delta_{01} + \Delta_{23} \quad (16)$$
$$(\Delta_{34} + \Delta_{25})1.25 = (\Delta_{24} + \Delta_{35}) 0.75 + \Delta_{23} + \Delta_{45} \quad (17)$$
$$(\Delta_{47} + \Delta_{56})1.25 = (\Delta_{46} + \Delta_{57}) 0.75 + \Delta_{45} + \Delta_{67} \quad (18)$$
$$(\Delta_{69} + \Delta_{78})1.25 = (\Delta_{68} + \Delta_{79}) 0.75 + \Delta_{67} + \Delta_{89} \quad (19)$$

Substitute in Eq. (16) the values of $E\Delta$ taken from Table III:

$$E(1250W + 250x + 250x)1.25 = E(-120x - 600W - 120x)0.75 + E(-600W - 120x - 576W - 192x - 192y)$$

Whence $$1117x + 192y = -3188.5W \quad (20)$$

Do likewise with Eq. (17), (18) and (19):

$$192x + 1192y + 160v = -1836W \quad (21)$$
$$160y + 1573.5v + 96z = -754.75W \quad (22)$$
$$96v + 686z = -2441W \quad (23)$$

If these equations are now multiplied by 4, their identity with Eq. (12), (13), (14) and (15) will be seen.

Thus it is evident that the results of the three methods are identical, and the reader may decide which method he prefers. As to the new method, however, a brief remark should be added here.

The Williot strain method follows directly from Hooke's law (unit stress is proportional to unit strain) combined with the geometrical principle involved in the Williot diagram. Little more can be said or written about it (which accounts perhaps for its simplicity in comprehension as well as in application) except that it is very flexible.

For example, suppose the intermediate verticals in Fig. 4 were omitted; one redundant member would remain. Compute the stresses and strains, assuming a stress of $5x$ in member 0-3, all members assumed in tension. Change the signs in all terms of Eq. (17) and (19), and add Eq. (16) to (19), inclusive, thereby eliminating Δ_{23}, Δ_{45} and Δ_{67}, leaving one equation for solving x.

Again, suppose that the top chords in the two end panels were inclined: Develop Eq. (16) and (19) from general Eq. (1) instead of Eq. (2).

A classic tower problem

The truss in Fig. 4 was used for comparative purposes and not because the treatment is considered necessary for a structure of this kind. In frames of this pattern the stress analysis is usually made by assuming either that the shear in any panel is equally divided between the two diagonals, or that the diagonals are to be designed as slender tension members incapable of carrying any compressive stress. Either assumption renders the truss statically determinate.

In order to show the adaptability of the Williot strain method to the solution of a more complicated problem, the structure shown in Fig. 5 has been chosen because it resembles on a small scale the towers of the George Washington Bridge. It has statically indeterminate reactions and redundant members. The stress analysis of these towers by the method of work has been clearly described in an excellent paper[1] by L. S. Moisseiff. One of his problems was the distribution of a portion of two equal vertical loads, centered on the inside columns, to the outside columns through the bracing system. This is a special case of loading, and in applying the Williot strain method to the tower a more general case of loading (Fig. 6) will be treated first.

The tower, which supports the nine W_1 *to* W_9 inclusive, is first divided into two parts by a vertical section through the center, and the interacting forces P, Q, V and R are supplied to replace the stresses in the cut members, all as represented by Fig. 6. The stresses in the redundant members are x, y, z, x', y', z', as marked. The four supports will be considered immovable, therefore the members 1-2 and 2′-1′ carry no stress, and each support has a horizontal reaction equal to the horizontal component of its diagonal.

It is clear from Fig. 6 that, whatever the external loads may be, the reactions at 1, 2, 2′ and 1′ and the stresses in all members may be statically determined in terms of the ten unknown stresses x, y, z, x', y', z', P, Q, V, R and the known loads W. The problem then is to derive ten independent equations. Seven such equations may be written, one for each panel, from Eq. (2), all stresses and strains considered positive, as follows:

$$(\Delta_{14} + \Delta_{23}) \sec\theta = (\Delta_{13} + \Delta_{24}) \tan\theta + 0 + \Delta_{34} \quad (24l)$$
$$(\Delta_{36} + \Delta_{45}) \sec\theta = (\Delta_{35} + \Delta_{46}) \tan\theta + \Delta_{34} + \Delta_{56} \quad (25l)$$
$$(\Delta_{58} + \Delta_{67}) \sec\theta = (\Delta_{57} + \Delta_{68}) \tan\theta + \Delta_{56} + \Delta_{78} \quad (26l)$$
$$(\Delta_{68}' + \Delta_{6'8}) \sec\phi = (\Delta_{68} + \Delta_{6'8'}) \tan\phi + \Delta_{66}' + \Delta_{88}' \quad (27)$$
Same as 26*l* except all subscripts primed (26*r*)
Same as 25*l* except all subscripts primed (25*r*)
Same as 24*l* except all subscripts primed (24*r*)

Four additional equations may be derived from the Williot diagrams for the three left panels, sketched separately for clearness in Fig. 7. Joints 1 and 2 are coincident, since there is no strain in member 1-2. The objective is to derive expressions for the horizontal displacements of joints 5, 6, 7 and 8, represented respectively by k, l, m and n. The following expressions are easily derived from Fig. 7:

$$a = \Delta_{23} \sec\theta - \Delta_{13} \tan\theta \quad (28)$$
$$b = \Delta_{14} \sec\theta - \Delta_{24} \tan\theta \quad (29)$$
$$c = \Delta_{45} \sec\theta - (\Delta_{13} + \Delta_{35} - \Delta_{24}) \tan\theta \quad (30)$$
$$d = \Delta_{36} \sec\theta - (\Delta_{24} + \Delta_{46} - \Delta_{13}) \tan\theta \quad (31)$$
$$e = \Delta_{67} \sec\theta - (\Delta_{13} + \Delta_{35} + \Delta_{57} - \Delta_{24} - \Delta_{46}) \tan\theta \quad (32)$$
$$f = \Delta_{58} \sec\theta - (\Delta_{24} + \Delta_{46} + \Delta_{68} - \Delta_{13} - \Delta_{35}) \tan\theta \quad (33)$$
$$k = c - b = (\Delta_{45} - \Delta_{14}) \sec\theta + (2\Delta_{24} - \Delta_{13} - \Delta_{35}) \tan\theta \quad (34)$$
$$l = d - a = (\Delta_{36} - \Delta_{23}) \sec\theta + (2\Delta_{13} - \Delta_{24} - \Delta_{46}) \tan\theta \quad (35)$$
$$m = e + a - d = (\Delta_{67} + \Delta_{23} - \Delta_{36}) \sec\theta + (2\Delta_{24} + 2\Delta_{46} - 3\Delta_{13} - \Delta_{35} - \Delta_{57}) \tan\theta \quad (36)$$
$$n = f + b - c = (\Delta_{58} + \Delta_{14} - \Delta_{45}) \sec\theta + (2\Delta_{13} + 2\Delta_{35} - 3\Delta_{24} - \Delta_{46} - \Delta_{68}) \tan\theta \quad (37)$$

Imagine all letters and numerals primed in Eq. (28) to (37) inclusive, and they will apply to the three right panels. It is evident that the horizontal displacements of joints 5 and 5′ must be accounted for by the strains in the horizontal members connecting them,

hence $k + k' = \Delta_{56} + \Delta_{66}' + \Delta_{5'6'}'$ (38)

also $m + m' = \Delta_{78} + \Delta_{88}' + \Delta_{7'8'}'$ (39)

likewise $l + l' + \Delta_{66}' = 0$ (40)

and $n + n' + \Delta_{88}' = 0$ (41)

The four equations in terms of the strains may now be written by substituting the expressions for k, k', m, etc., from Eq. (34) to (37) inclusive and (34′) to (37′) inclusive.

Apparently we have eleven equations, (24*l*) to (24*r*) inclusive and (38) to (41) inclusive, for the solution of ten unknowns. One equation, however, is not independent. Eq. (38) to (41) inclusive take into account the actual and therefore relative displacements of joints 5, 6, 7, 8, 5′, 6′, 7′ and 8′. The relative displacements of these joints are also accounted for by Eq. (26*r*) and (26*l*); therefore one of these six equations is superfluous. It will be seen on inspection that the sum of Eq. (38) to (41) is the same as the sum of Eq. (26*r*) and (26*l*). Each group says the same thing. Any one of these equations may be derived from the remaining five,

TABLE I—STRESS CALCULATION BY METHOD OF WORK, FOUR-PANEL TRUSS

Member	$\frac{l}{A}$	S'	w_{03}	w_{25}	w_{56}	w_{78}	$\frac{l}{A} S'w_{03}$	$\frac{l}{A} S'w_{25}$	$\frac{l}{A} S'w_{56}$	$\frac{l}{A} S'w_{78}$	$\frac{l}{A} w^2_{03}$	$\frac{l}{A} w^2_{25}$	$\frac{l}{A} w^2_{56}$	$\frac{l}{A} w^2_{78}$	$\frac{l}{A} w_{03}w_{25}$	$\frac{l}{A} w_{03}w_{56}$	$\frac{l}{A} w_{03}w_{78}$	$\frac{l}{A} w_{25}w_{56}$	$\frac{l}{A} w_{25}w_{78}$	$\frac{l}{A} w_{56}w_{78}$
1—3	40	−15W	−.6				+ 360W				+ 14.4									
3—5	20	−24W		−.6				+288W				+7.2								
5—7	15	−24W			−.6				+216W				+5.4							
7—9	30	−21W				−.6				+378W				+10.8						
0—2	40	0	−.6				0				+14.4									
2—4	20	+15W		−.6				−180W				+7.2								
4—6	15	+21W			−.6				−189W				+5.4							
6—8	30	0				−.6				0				+10.8						
1—2	50	+25W	+1.				+1250W				+50.									
3—4	60	+15W		+1.				+900W				+60								
4—7	100	+ 5W			+1.				+500W				+100							
6—9	30	+35W				+1.				+1050W				+30						
0—3	50	0																		
2—5	60	0																		
5—6	100	0																		
7—8	30	0																		
0—1	30	−20W	−.8				+480 W				+19.2									
2—3	48	−12W	−.8	−.8			+460.8W	+460.8W			+30.72	+30.72			+30.72					
4—5	40	0		−.8	−.8			0	0			+25.60	+25.60					+25.60		
6—7	24	− 4W			−.8	−.8			+76.8W	+76.8W			+15.36	+15.36						+15.36
8—9	20	−28W				−.8				+448.W				+12.80						
						Σ =	+2550.8W	+1468.8W	+603.8W	+1952.8W	+128.72	+130.72	+151.76	+79.76	+30.72	0	0	+25.60	0	+15.36

TABLE II—STRESS CALCULATION BY LEAST WORK, FOUR-PANEL TRUSS

Member	$\frac{l}{2A}$	S	$\frac{S^2l}{2A}$											
1—3	20	−15W −3x	4500W²	+ 1800Wx	+ 180x²									
3—5	10	−24W −3y	5760W²			+1440Wy	+ 90y²							
5—7	75	−24W −3x	4320W²					+1080Wx	+67.5x²					
7—9	15	−21W −3x	6615W²							+1890Wx	+ 135x²			
0—2	20	−3x			+ 180x²									
2—4	10	15W −3y	2250W²			− 900Wy	+ 90y²							
4—6	75	21W −3x	3308W²					− 945Wx	+67.5x²					
6—8	15	−3x									+ 135x²			
1—2	25	25W +5x	15625W²	+6250Wx	+ 625x²									
3—4	30	15W +5y	6750W²			+4500Wy	+ 750y²							
4—7	50	5W +5x	1250W²					+2500Wx	+1250x²					
6—9	15	35W +5x	18375W²							+5250Wx	+ 375x²			
0—3	25	5x			+ 625x²									
2—5	30	5y					+ 750y²							
5—6	50	5x							+1250x²					
7—8	15	5x									+ 375x²			
0—1	15	−20W −4x	6000W²	+ 2400Wx	+ 240x²									
2—3	24	−12W −4x −4y	3456W²	+ 2304Wx	+ 384x²	+2304Wy	+ 384y²							
4—5	20	−4y −4x					+ 320y²		+ 320x²			+768xy		
6—7	12	− 4W −4x −4x	192W²					+ 384Wx	+ 192x²	+ 384Wx	+ 192x²		+640yz	
8—9	10	−28W −4x	7840W²							+2240Wx	+ 160x²			+384xz
$\sum \frac{S^2l}{2A}$ =			86246W²	+12754Wx	+2234x²	+7344Wy	+2384y²	+3819Wx	+3147x²	+9764Wx	+1372x²	+768xy	+640yz	+384xz

is a dependent equation and may be omitted.

It may be clarifying to some readers to develop the relations between the two parts of the structure in a slightly different manner. Instead of expressing the movements of joints 5 and 5′ as in Eq. (38), and the movements of joints 7 and 7′ as in Eq. (39), we might consider the movements of joints 6 and 8′ with respect to the strain in the diagonal connecting them, and give a similar treatment to the other diagonal in the center panel, as follows:

By hypothesis all members are treated as if in tension; hence the vertical movement of joint 8′ with respect to joint 6 is,

$$E = \triangle_{2'4'} + \triangle_{4'6'} + \triangle_{6'8'} - \triangle_{24} - \triangle_{46}$$

In Fig. 2*b*, E corresponds to the vertical distance from A to D, which is,

$$f - h = \triangle_{dd} \operatorname{cosec} \theta - \triangle_{ac} \cot \theta$$

Hence, by substituting the homologous subscripts, and also replacing θ by ϕ,

$$E = f - h = \triangle_{68'} \operatorname{cosec} \phi - \triangle_{66'} \cot \phi$$

whence,

$$\triangle_{2'4'} + \triangle_{4'6'} + \triangle_{6'8'} - \triangle_{24} - \triangle_{46} = \triangle_{68'} \operatorname{cosec} \phi - \triangle_{66'} \cot \phi \qquad (A)$$

In similar manner, by considering the vertical movement of joint 8 with respect to joint 6′ we have,

$$\triangle_{24} + \triangle_{46} + \triangle_{68} - \triangle_{2'4'} - \triangle_{4'6'} = \triangle_{6'8} \operatorname{cosec} \phi - \triangle_{88'} \cot \phi \qquad (B)$$

Eq. (24*l*) to (24*r*), taken with Eq. (40), (41), (*A*) and (*B*), apparently give eleven equations for solving ten unknowns; but, as might be expected, Eq. (27), (*A*) and (*B*) are not three independent equations. For, by adding Eq. (*A*) and (*B*) we have,

$$\triangle_{68} + \triangle_{6'8'} = (\triangle_{68'} + \triangle_{6'8}) \operatorname{cosec} \phi - (\triangle_{66'} + \triangle_{88'}) \cot \phi$$

and on multiplying both sides of this equation by tan ø we obtain Eq. (27).

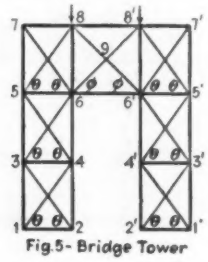

Fig.5- Bridge Tower

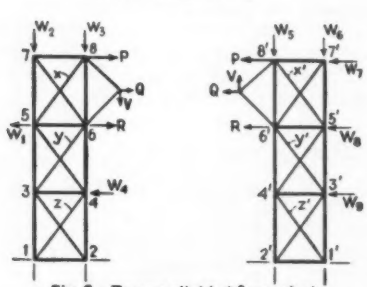

Fig.6 - Tower divided for analysis

FIGS. 5 AND 6—BRIDGE TOWER analyzed by Williot strain method—a structure of the general type of the George Washington Bridge towers.

TABLE III—STRESS CALCULATION BY WILLIOT STRAIN METHOD. FOUR-PANEL TRUSS

Member	$\frac{l}{A}$	S	$E\triangle = \frac{Sl}{A}$
1—3	40	$-15W - 3x$	$-600W - 120x$
3—5	20	$-24W - 3y$	$-480W - 60y$
5—7	15	$-24W - 3v$	$-360W - 45v$
7—9	30	$-21W - 3z$	$-630W - 90z$
0—2	40	$-3x$	$-120x$
2—4	20	$15W - 3y$	$300W - 60y$
4—6	15	$21W - 3v$	$315W - 45v$
6—8	30	$-3z$	$-90z$
1—2	50	$25W + 5x$	$1250W + 250x$
3—4	60	$15W + 5y$	$900W + 300y$
4—7	100	$5W + 5v$	$500W + 500v$
6—9	30	$35W + 5z$	$1050W + 150z$
0—3	50	$5x$	$250x$
2—5	60	$5y$	$300y$
5—6	100	$5v$	$500v$
7—8	30	$5z$	$150z$
0—1	30	$-20W - 4x$	$-600W - 120x$
2—3	48	$-12W - 4x - 4y$	$-576W - 192x - 192y$
4—5	40	$-4y - 4v$	$-160y - 160v$
6—7	24	$-4W - 4v - 4z$	$-96W - 96v - 96z$
8—9	20	$-28W - 4z$	$-560W - 80z$

The problem is much simplified when the loads are vertical and symmetrical as, for example, in Fig. 5. There is no vertical shear in the center panel; $V = 0$; joint 9 and the mid-points of members 6-6′ and 8-8′ deflect in a vertical line. Eq. (38), (39), (40) and (41) then become,

$$k = \triangle_{56} + \tfrac{1}{2}\triangle_{66'} \qquad (42)$$

$$m = \triangle_{78} + \tfrac{1}{2}\triangle_{88'} \qquad (43)$$

$$l + \tfrac{1}{2}\triangle_{66'} = 0 \qquad (44)$$

$$n + \tfrac{1}{2}\triangle_{88'} = 0 \qquad (45)$$

We have also Eq. (24*l*), (25*l*), (26*l*) and (27), making eight in all for the solution of six unknowns x, y, z, P, Q and R. Two of the eight are not independent equations. Eq. (24*l*) and (25*l*) taken together fix the relative displacements of joints 5 and 6, which are also fixed by Eq. (42) and (44), hence any one of these four equations may be taken as one of the dependent equations, since it may be derived from the remaining three. Likewise Eq. (24*l*), (25*l*) and (26*l*) taken together fix the relative displacements of joints 7 and 8, which are also fixed by Eq. (43) and (45); therefore any one of these five equations may be taken as the other dependent equation. On the other hand, all eight equations might be used, the identities permitted to show themselves in the solution, and serve as a partial check on the work.

The trigonometric functions occupy considerable space in the equations and perhaps lend a formidable appearance, but it should be remembered that they all have known numerical values which may be substituted when the equations are being written. If the panels vary in height, θ will vary accordingly, but this does not complicate Eq. (24*l*) to (24*r*) at all. In Eq. (28) to (33) inclusive the quantities $\frac{Sl}{AE}$ may be substituted for the $\triangle$ values in each equation before the expressions for k, l, m and n are written. If the outside columns are battered, then Eq. (24*l*) to (24*r*), except (27), should be developed from Eq. (1); if all columns are battered, as in a two-column tower, then Eq. (3) should be used.

Where flexure is involved

The Williot strain equations reflect their brightest light when seen in conjunction with the method of expressing end moments in terms of angular displacements as represented by the general equation:

$$M_{ab} = \frac{2EI}{l}\,(2\phi_a + \phi_b - 3\psi_{ab})$$

in which ϕ_a and ϕ_b represent the angular rotations of the joints A and B, and ψ_{ab} represents the angular rotation of the member AB. When AB is any member of a truss, ψ_{ab} may be determined from a Williot diagram drawn after the stresses and corresponding strains have been computed. But suppose that the stresses are statically indeterminate because of some peculiarity in the structure: imagine, for example, a truss with chord members so stiff that the almost universal assumption of all shear being resisted by the web system, whether single or double, is obviously absurd. We are confronted with the problem of finding the distribution of shear in each panel between its chords and webs. If one insists that the methods of work and least work are applicable, one also should admit that either method is cumbersome. There are, of course, several makeshift methods of cut-and-try or make-believe which might be used, with varying degrees of inaccuracy.

But even when the stresses are unknown, a Williot diagram may be sketched, and expressions for ψ in terms of the unknown strains (and therefore stresses) developed from the geometry

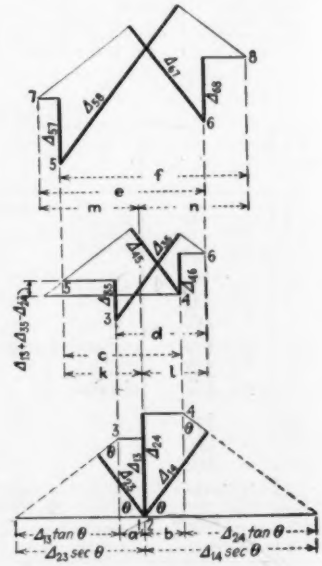

FIG. 7—SPECIAL Williot diagrams for additional equations required in bridge-tower problem.

of the Williot diagram. An equation $\Sigma M = 0$ may then be written for each joint, the quantities ϕ and ψ determined, and finally the moments at the ends of each member (and hence the shear in each member) expressed in terms of the unknown stresses. The principles of statics will do the rest.

It was during the search for a rational solution to a problem of this nature that the Williot strain method was discovered (*EN-R*, Jan. 25, 1934, p. 125). The steps leading up to the discovery and the application have been treated in another monograph (*Proc.*, Am.Soc. C.E., January, 1934). In the present paper the writer has attempted to explain the new method, to compare it with two old and well-tried methods in the solution of a simple problem and to show its practibility by outlining its application to a more advanced structure. He has not yet attempted to investigate its possible range of application, but feels confident that the profession will find many ways in which to utilize the Williot Strain Method to advantage.

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Uniform Traffic Signals for Illinois Roads

New state regulations for traffic-control signals apply to state roads in municipalities—General requirements summarized

A POLICY governing traffic-control signals on state highways has been declared by the Illinois division of highways. This action has been taken in accordance with powers granted in recent amendments to existing road laws. These new powers extend to state roads within municipalities; and the new regulations, which are in line with the general recommendations of the National Conference on Street and Highway Safety, are directed particularly to practices in municipalities.

Cities must secure a permit from the state highway department to install, operate and maintain traffic-control signals on state highways within their boundaries. Plans, showing the location of the signal units, pipes, conduits or cables, and all other appurtenances such as the controller and the service outlet must accompany the request. This requirement covers fixed-time and traffic-accentuated stop-and-go signals, flashing signals, stated-speed signs, stop signs and all cautionary signs. All such existing signs not warranted in the opinion of the state officials must be removed. The state will insist on a reasonable warrant for every signal.

As an indication, stop-and-go signals are advantageous in some instances, but in many other situations there is no legitimate warrant for their use. They may cause undue delay, incite motorists to pass the lights, breed disrespect for all signals, and they may actually increase accidents, as before-and-after records have shown. For these reasons the state insists that it have reasonable assurance that traffic facts warrant signals. It requires proper timing and operation best to suit traffic fluctuations by hours, days of the week and seasons. Type and design (not make) are required to be in accord with accepted standards.

The necessity of signals is determined by prevailing volume of vehicular and pedestrian traffic and by accident frequency. Usually some peak flow is pointed to as justification for stop-and-go regulation. A Fourth of July or Labor Day peak is not considered to justify a signal to be operated the year round. Neither do conditions occurring only one day a week. Traffic peaks warranting a signal should be recurring more than once a week and also should have reasonable duration. Where such is not the case, part-time police control is often found more economical and more satisfactory.

Before the state will grant authority to erect a stop-and-go signal, it will require actual figures determined by traffic counts made by the municipality, the state or some other recognized authority. If accidents, either alone or in combination with the quantity of traffic, are believed to be sufficient, then adequate records of all accidents over a reasonable period of time will have to be submitted as evidence. The record of each accident should be sufficiently complete to indicate whether or not the presence of the signal would have prevented the accident.

Operation of signals

Traffic facts are also necessary to know how to vary the operation of a signal. The length of cycle must be determined, also how much time shall be allotted to each street. But more important is the determination of when a signal shall be operated as a stop-and-go device and when as a warning signal. It is considered that small-city, town or suburban traffic conditions seldom warrant the continuous operation of stop-and-go control throughout the day and evening. It is possible that such control may be justified during a period of heavy flow that may be a daily occurrence; at all other times traffic can be better served without stop-and-go regulation. To shut off signals entirely or to turn on a small light on top of the signal during the light traffic periods, however, is not considered satisfactory. A much better way, the department holds, is to change the operation of the signal from a stop-and-go device to a flashing-warning device.

Depending on the flow of traffic at each particular corner, three variations of this flashing warning device are possible: flashing yellow lights showing on both streets, flashing red lights on one street and yellow on the other, and flashing red lights on both streets. The meaning of these flashing or "beacon" lights, as they are sometimes called, is established and standardized nationally: red, stop and proceed when safe; yellow, slow speed and proceed with caution. New signals can be bought to permit such a change in operation, and old signals can be readily adapted.

A definite schedule for changing the operation of signals for each day of the week and season of the year can easily be worked out after traffic counts have been made. Municipal police or other officials can then be charged with the duty of changing the operation according to the schedule.

When a road or street has two or more signals fairly close together, it is essential that they be interconnected and that their operation as stop-and-go signals be coordinated. By so doing delay to traffic will be kept to a minimum, and vehicles will be held to a speed determined by the timing of the lights. Signals actuated by traffic approaching the intersection may be desirable under some conditions or permissible at intersections where traffic does not warrant a fixed-time signal.

To justify the installation and operation of traffic-control signals, the total volume of traffic passing through an intersection during the maximum hour

the regulations state, shall be 1,000, 1,400 and 1,800 respectively, for one, two and three lanes in each direction. The total traffic on the less-traveled thoroughfares during the maximum hour shall not be less than 30 per cent of these figures. Heretofore this figure has usually been 25 per cent. Special considerations such as accident hazard, the amount of pedestrian traffic, visibility in each direction, gradients, railroad and street-car traffic and coordinated movement along thoroughfares shall be considered; and in some instances may be sufficient justification for deviation. When the total vehicular volume entering an intersection having fixed time signals falls below 500 vehicles per hour for a period of two hours or more the rules require that the fixed-time signals shall be operated as a caution or a stop-and-go signal.

Vehicle-actuated signals are not subject to the above minimum reuirements and may be installed at intersections (where conditions warrant) that are independently controlled or where in the opinion of the department, it will not become necessary within the near future to incorporate such an installation in a progressively operated system of signals along the thoroughfare. Vehicle-actuated signals, according to the rules, should be considered at all intersections where (*a*) there is a marked irregularity during the day in the ratio of main thoroughfare traffic to traffic on the less-traveled thoroughfare; and (*b*) there are special physical or traffic conditions which, in the opinion of the department, render a signal of another type unsuitable.

Fixed-time controllers of all types should be so designed that a timing schedule for any normal traffic condition can be easily set up. The flexibility of controllers for use at the normal intersection of two streets should make it possible: *(a)* to set up at least six separate intervals or periods; *(b)* to vary the length of the total cycle between 30 and 180 sec. by steps not to exceed 15 per cent; (*c*) to vary the length of intervals or periods independently by small steps to cover virtually any set of timing requirements. Time cycles are to be as short as possible and will be based on the longest sustained hourly average volume of traffic entering the intersection from each direction with consideration of the discharge capacity in each direction.

All traffic-control signals within 1,500 ft. of each other are to be operated by progressive coordinate control. The department may require progressive coordinate operation of signals located within one-half mile of each other if, in the opinion of the engineers, such operation would improve the movement of vehicles along the thoroughfare. A series of traffic-control signals along a thoroughfare is to be operated as a progressive or a flexible progressive system. All systems of traffic-control signals are to operate so as to permit a continuous movement of traffic through the controlled area at a determined rate of speed suitable for the area so controlled. Signs are to be erected on the signals indicating the speed for which the signals are set.

All traffic-control signals installed within 150 ft. of a railroad grade-crossing are to be interconnected with the railroad warning device in such manner as to give automatically a stop indication for cross-track traffic when a train is approaching.

Regulations as to time and place of parking on state highways within municipalities rest with the state, which may prohibit entirely. Appropriate signs indicating such regulations are to be posted by the department or by the municipality with the approval of the state, which may remove signals and signs not conforming to the law.

Metal Arch Culvert Carried by Timber Mat Floating on Muck

IN DESIGNING a state-aid highway in northeastern Massachusetts, the state department of public works recently found that not only had a wooden bridge top passed its period of usefulness, but both abutments, of loose stone, were sliding and overturning. Investigation disclosed a depth of several feet of soft muck, which for the usual type of rigid structure would have increased the cost of foundations out of all proportion.

A preliminary study indicated that a considerable saving could be made by using a semi-flexible structure floated on the muck. This construction was decided upon, and plans were made for a multi-plate corrugated metal arch of 120-in. span and 62½-in. rise on a heavy timber mat. Further to reduce eccentric loadings on the very unstable foundation material, the plans eliminated headwalls by cutting the ends of the metal arch to a 1-on-1 slope, the embankment slopes to be stone-paved.

During construction it early developed that, because of soft muck and quicksand, excavating would probably be difficult. It was decided that, to save time and to avoid keeping the excavation open any longer than necessary, the timber mat should be constructed complete on an area outside the highway limits. Work on the excavation and the construction was finished, the completed timber mat was placed as a unit, and the corrugated metal arch was erected on it. Both arch and mat are semi-flexible and are strongly bolted together. All open spaces below the floor and between the floor members were tamped full of selected gravel.

All work was performed by the town of Boxford under the direction of the Beverly district office of the Massachusetts department of public works.

TIMBER MAT (left) and corrugated metal arch carry roadway over soft muck on Massachusetts highway.

Construction Progress on the Golden Gate Bridge

Both anchorages ready for the cables—North tower, 702 ft. high above the pier, nearing completion—Future progress depends on foundation work on south pier, going forward under severe difficulties

FEATURING the record-breaking 4,200-ft. suspension span over the entrance to San Francisco Bay, the Golden Gate Bridge project is now more than 25 per cent completed. Both anchorages have been finished ready for the attachment of the cables, the two pylons (cable bent piers) are approaching completion, the north (Marin County) pier has been finished, and tower erection has advanced more than two-thirds of the total height of 746 ft. (including the 40-ft. height of the pier). The key to construction progress centers on the foundation at the south (San Francisco) pier site, 1,000 ft. offshore in the swift tide and stormy waters of the Golden Gate, where the difficulties equal, if not surpass, any other bridge-foundation undertaking. The present construction program indicates a completion date in the first part of 1937.

The project is being carried out by the Golden Gate Bridge & Highway District, a political subdivision consisting of the city and county of San Francisco and all or parts of six counties lying north of the Golden Gate. The bridge will be a publicly owned toll structure. The district was organized following enabling legislature in 1923. A $35,000,000 bond issue was voted in November, 1930, and after considerable delay resulting from legal questions the district secured initial financing with the sale of $6,000,000 of bonds to a local syndicate in November, 1932. In the meantime the original construction contracts had lapsed, and new bids were called on several of the schedules. These new contracts were awarded November, 1932, and construction started immediately. A more extended review of the project, including history and the principal features of the original design, appeared in *Engineering News-Record*, May 28, 1931, p. 890.

Briefly, the principal features of the project (Fig. 1) consist of a 4,200-ft. suspension span and two 1,125-ft. suspended side spans. The two parallel-wire cables will be 36¼ in. in diameter and will be spaced 90 ft. apart, providing a 60-ft. roadway and two 11-ft. sidewalks. Vertical clearance at the center of the span will be 220 ft. above mean lower low water with a clearance of 210 ft. near the piers. The steel towers are 702 ft. high above the tops of the concrete piers at El. 44. Further details of the superstructure design will be covered by subsequent articles.

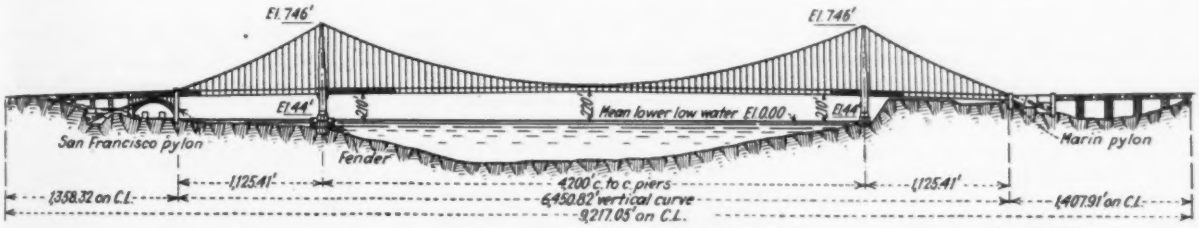

FIG. 1—GOLDEN GATE BRIDGE, showing general features of project, principal dimensions and foundation-rock profile.

As the result of the separate contracts let for the two piers and the two anchorages, each contractor had to place concrete on both sides of the mile stretch of rough water. The logical result was the letting of a joint subcontract for furnishing concrete on both sides of the site for these two main contracts as well as for minor work. The concrete subcontractor erected a modern unloading and batching plant at water's edge on both sides of the Gate. Both plants included facilities for unloading aggregate and bulk cement from barges, a storage and a modern batching plant equipped with the latest control devices. Each plant has a capacity of 160 cu.yd. of batched aggregates per hour, which is dumped into 4-yd. mixer-body trucks and moved to the various places of pour. This set-up has proved successful in placing 120,000 cu.yd. of concrete to date in the two anchorages and the north pier.

The cable anchorages (Fig. 2) are of the gravity type. The block for each cable end is about 60 ft. wide, 170 ft. long and of varying depth, containing 32,000 cu.yd. of concrete in which are embedded 61 pairs of eyebar chains of three links each terminating in anchorage girders. The anchorages are bedded into solid rock and are designed with stability sufficient to develop the full strength of the cables. The first stage of construction has been completed, and they are now in readiness for the cables.

A reinforced-concrete pylon is provided at the shore end of each side span to support the side span, tie down the cables and receive the lateral loads from the cables and the side span. Each pylon comprises two hollow columns. The pylon on the San Francisco side is founded on rock at El. —10 and, when completed, will rise to El. 250. The pylon on the Marin side is founded on rock at El. 60 and will extend to El.244.

North pier

The north pier location is a ledge of rock near the water's edge at the base of a 200-ft. cliff. This natural site, where borings indicated a compact greenstone or diabase, did not involve serious foundation problems. A steel sheetpiling and earth-dike cofferdam, with a few timber cribs at the lower corner, was placed around three sides of the pier site, using the natural cliff for the fourth side. The site was unwatered, and good foundation rock was found over the 80x160-ft. area at El. —20, with this depth increased to —32 ft. as the maximum at the southwest corner. The base block extending up to El. —10 is surmounted by the pier, which extends to El. 44, with a section decreasing from 72x149 ft. at the base to 65x134 ft. at the top. The pier was divided up into eleven sections for concreting, and an elaborate system of horizontal and vertical keys was provided. Concrete was placed with a 100-ft. steel mast hoisting a 1-yd. bucket, which discharged through counterweighted chutes over the entire area of the pier.

An outstanding construction problem included the erection of a 180-ton mat of reinforcing steel near the top of the pier, to distribute the concentrated tower loads over the entire area. This mat included four layers of 1¼-in. square bars and had to be supported by a structural-steel frame 30 ft. in the air prior to the concreting of the last lift of the pier. Work on the Marin County pier began in January, 1933, and it was finished in June.

Erection of the steel tower that extends from the top of the pier at El. 44 to the highest point at El. 746 began

Nov. 8, 1933, and more than two-thirds of the 22,000 tons of steel has been erected (Fig. 3). The design of this tower and the erection procedure will require subsequent detailed treatment.

South pier

The south pier is founded on rock that consists of an aggregate of massive spheroids of strong serpentine embedded in a matrix of sheared serpentine. This formation slopes gradually down from the shore line for a distance of more than 1,000 ft. and then drops off at a faster rate (Fig. 1). At a point 1,000 ft. from shore the water depth is about 65 ft. and there is no overburden, the tidal flow keeping the rock clean. The selection of a 4,200-ft. span, which resulted partly from these foundation conditions, placed the south pier location about 1,000 ft. from shore in the 65-ft. water depth. To decrease the span would have moved the south pier farther offshore with the depth of water increasing rather rapidly.

Exploratory drilling at the site was described in *ENR*, April 10, 1930, p. 614, and July 17, 1930, p. 92. Supplementing these original borings and soundings, an extensive survey has been completed recently which covered an area of about 10 acres around the pier site. The very accurate soundings thus obtained verified the soundings previously taken, and conclusively established the contours of the rock floor for a distance 400 ft. out from the pier in all directions.

Original program

From the outset the difficulties of founding the pier in 65 ft. of swift water indicated the advantages to be gained by building the final fender system first, to permit the sinking of the pier caisson, as a subsequent step, in the quiet water. The design calls for an elliptical concrete-fender wall surrounding the pier site with maximum diameters of 155 and 292 ft. This concrete fender wall will be 27½ ft. thick and will extend from foundation on rock at El. —98 to El. +15 ft. The construction program provided for building the fender in horseshoe shape, floating in the caisson, completing the fender and finally sinking and sealing the caisson against the rock under air. This constituted the construction procedure as originally outlined.

It was realized that this unit would present construction difficulties and problems probably unsurpassed in bridge-foundation work. At the present time the work has advanced, after some delay because of accidents and changes in method, to the point where the fender system is about to be started. The details of this procedure, the construction equipment, operations and the methods have not been completed, but the process as outlined is the one that has been adopted.

Excavation—The first two operations of the contractor were to construct a trestle to the pier location (Fig. 4) and excavate the site. The original plan contemplated the sinking of the pier itself into rock, with the founding of the fender system approximately on rock surface. Sinking the pier into rock is intended to assist in stabilizing it against earthquake movements and to deliver the load to the foundation at a depth where the rock is well confined on all sides. This method contemplated the excavation of all rock under the 90x185-ft. caisson by the pneumatic process. The high unit cost of this excavation and the length of time required resulted in the contractor changing the program to excavate by subaqueous operations the entire pier and fender site as close to the final foundation elevation as possible and thereby reduce to a minimum the amount of excavation to be carried on under air.

This changed procedure proved acceptable to the district and was carried forward by using a unique process of subaqueous blasting. By the dropping of small bombs against the rock, holes about 20 ft. deep were shattered, and large bombs were driven into these and exploded to loosen the rock. The details of this bombing operation were described in *Engineering News-Record* July 27, 1933, p. 93. The process proved very successful in the removal of rock over the entire pier and fender area, and the operations were stopped at an average depth of El. —95.

In the meantime the steel and timber trestle had been constructed to the site from the shore. The legs of this trestle were set in rock by the same principle of bombing, and the details of its design and construction were described in the same article. Some time after the trestle had been completed it was rammed by a ship during a heavy fog, but the damage was only slight. In December, 1933, during a severe storm, waves battered the trestle to such an extent that a length of 700 ft. at the outer end was carried away. At that time the trestle had been completed to the actual fender site, and fender construction had been started.

A heavy steel frame, referred to as a guide frame, had been erected at the end of the trestle, and this was being used to sink the sectional parts of the structural frame that provided the form

FIG. 2—GRAVITY ANCHORAGE at the north end of the bridge ready for cable attachment. Each of the two blocks contains 61 pairs of eyebar chains to which the end of the 36¼-in. cable will be fastened.

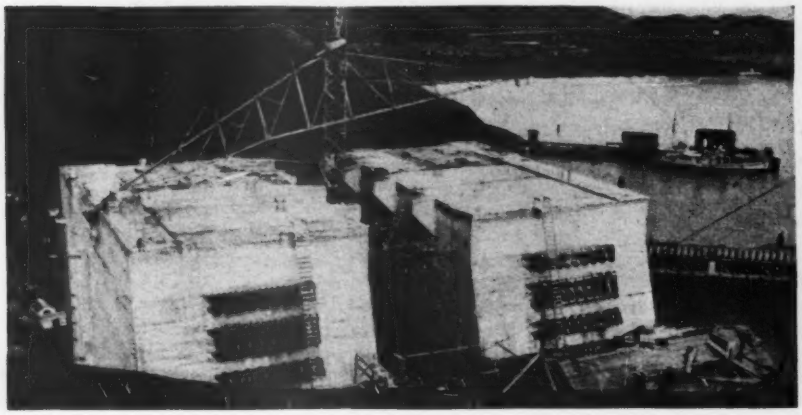

FIG. 3—NORTH TOWER erected to about two-thirds of its total height of 746 ft. above water and 702 ft. above concrete pier top. The view looks southeast, with San Francisco across the Golden Gate.

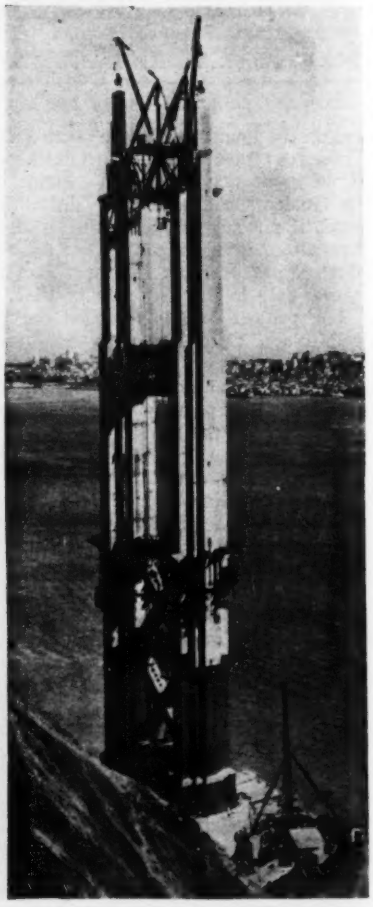

FIG. 4—A TRESTLE 1,000 ft. long provides access to the south pier site. A derrick barge is shown at the pier site excavating rock loosened by subaqueous blasting. At the left is the cable bent pier (pylon) and old Fort Scott, which is being maintained for historic reasons. Across the mile stretch of swift Golden Gate water can be seen the north tower under erection. The space between the derrick barge and the north tower corresponds to the 4,200-ft. main span of the bridge.

for concreting the first unit of the fender system. At that time the plans for the construction of the fender system provided for erecting it in sections 27½ ft. thick by 30 ft. long (direction of the line of fender), and for the total height of about 80 ft. Each of these fender sections was to consist of a structural-steel frame to support forms on the sides that would permit the placing of the concrete for the fender. The guide frame was designed to assist in the setting of the first section, and the subsequent sections would be set in position using an interlocking device, building out from the initial unit. After the accident resulting from the storm, the entire construction problem was restudied and a change made in the general procedure.

Plans modified

As a prerequisite, the trestle has been reconstructed, using much heavier design with seven-pile bents and brace piers, or towers, at every fourth bent. Longitudinal stiffness has been provided by cables running diagonally from caps to piles, and also the entire trestle has been stabilized by anchors placed on both sides. A further precaution was to raise the deck elevation 5 ft., which eliminates much of the danger from the force of wave uplift. This work is about completed, and the trestle end is at the fender line.

It was decided to deepen the fender excavation to provide about a 20 -to 25-ft. surcharge of rock entirely around the site. This required the removal of about 20,000 cu.yd. of additional rock. Excavation has been carried out while the trestle was being rebuilt. In its present form the entire pier and fender site presents a rock surface varying from El. —95 to —100.

As the result of the experience in attempting to sink the first fender section, the program for fender construction was changed. In place of vertical units extending all the way from rock to the final elevation, the modified plan is to build up the fender wall in lifts of 20 to 40 ft. In this procedure the first fender sections will raise and complete the fender to El. —60 before the next lift is started. This operation will be carried out by sinking into position timber bulkheads and structural-steel frames, with steel plates along the sides above El. —80 to provide the forms for tremie-concreting. After this first lift has been completed, the second lift will be placed, consisting of similar units, to raise the wall to El. —40. Then, all but the easterly end will be continued up in the dry to protect the site from the sea side, and the caisson will be floated into position. The fender will then be completed.

The caisson will be sunk by the usual process of building up the concrete until it comes to rest on a ledge on the inside of the fender system at —80 ft. At this stage a concrete seal will be effected in the space between the pier and the fender. Depending upon the effectiveness of this seal and the contact of the fender system with the rock, the area under the caisson will be unwatered, using as much air pressure as necessary. This will permit the site to be examined and such excavation to be carried out to deepen the rock as may be required. The last step will be to place concrete between the final rock elevation and the bottom of the caisson at El. —80, after which the pier will be completed to its final height at El. 44.

Organization — The engineering organization of the Golden Gate Bridge and Highway District is headed by Joseph B. Strauss, chief engineer. Clifford E. Paine is principal assistant engineer, and Russell G. Cone resident engineer. Collaborating with the chief engineer is a consulting board composed of O. H. Ammann, Chas. Derleth, Jr., and Leon S. Moisseiff.

The Pacific Bridge Co. has the contract for the two main piers, and Barrett & Hilp has the contract for the anchorages. McClintic-Marshall Corp. holds the contract for structural-steel work of the main structure. John A. Roebling's Sons Co. is the contractor for the cables and suspenders.

Paint for Water-Tank Interiors

Tests of 196 different kinds of paints and other coating materials for the protection of inside surfaces of steel water tanks have recently been completed by the Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa. Paints and coatings were applied in vertical panels to the interior surface of an open-top, 1,000,000-gal. tank owned by the Borough of Ambridge, Pa. After exposure for a period of 571 days, during which time the tank was kept in service, except when it was drained for the purpose of inspection, the various paints and coating materials were rated by two engineers. Copies of the complete report can be obtained from the Pittsburgh-Des Moines Steel Co., Neville Island, Pittsburgh, Pa.

Engineering Laboratory Problems Discussed at Iowa Meeting

PROBLEMS of educational laboratories provided the theme for discussion at the meeting of 150 professors and instructors from ten engineering colleges at the University of Iowa, April 13 and 14. The occasion was an annual open-house conference of midwest engineering educators, started many years ago by the two Iowa state schools, and subsequently joined by the Universities of Minnesota, Wisconsin and Illinois. This year it included Armour Institute of Technology, Washington University, Cornell College and the state institutions of Missouri and Nebraska. Laboratories were discussed as to their function in engineering education, the curriculum problems, procedure, scope and technique, with attention to special hydraulic laboratory problems of silt, river control, pneumatics and micro-motion studies. The meeting provided an opportunity for the inspection of the hydraulic laboratory and the new mechanical engineering laboratory.

New responsibilities

The beginning of a new era in engineering education, which is tending to fit the graduate to the changing economic status of the country, was discussed in the introductory remarks of Dean C. C. Williams, University of Iowa. In his opinion, this change required a re-appraisal of the problem of how to get the most out of laboratory inspection within the financial limit of the engineering schools and the time schedules of the students. The elaborately equipped hydraulic laboratory at Iowa City seemed to him to be a public obligation, because of the location of the university on the river, where these facilities could be so easily developed. In the design of the new mechanics laboratory, and particularly in its equipment, attention has been paid to the modern requirements of industry which have been satisfied by a section on pneumatics and a manufacturer's laboratory, including elaborate equipment for micro-motion study.

President A. C. Willard, University of Illinois, until recently dean of the college of engineering, introduced a discussion-provoking subject on the general status of the undergraduate laboratory and its curricula requirements, by going back fully a century to the genesis of the laboratory method of teaching established by Amos Eaton at Rensselaer, where lecture and experiment by the student rather than the professor ushered in a new era in the field of scientific teaching. At the present time the University of Illinois is conducting an extensive review of its 89 laboratory courses through a questionnaire to the students, in an effort to determine what they "pay" in time and effort for the credit received.

Basic quantitative results are expected, and some were listed tentatively. It is evident that with an engineer at the head of the University of Illinois there will be a re-appraisal of all its instruction with definite objectives and points measurable by some yardstick of accomplishment. A laboratory course, in the opinion of President Willard, primarily should teach scientific methods, give background, stimulate curiosity and provide a new basis for personal contact between professor and student. Some less essential though desirable things—and these were all vigorously discussed later—were the determination of accepted information obtainable in handbook form, routine tests, finished drawings and English.

As to English, particularly pertinent discussion developed its highly essential character. At Iowa State College the English department follows the progress of all engineering students in the three upper years, requiring the submission of a sizeable report each year, which is returned with comment and criticism. About 3,000 such papers are handled. If the quality of any paper is below standard, more English is required before graduation. Prof. M. P. Cleghorn, Ames, is an exponent of small laboratory groups, 5, 6 or 7 at most, to provide personal contacts and give responsibility to one of the students in charge.

Modern hydraulic laboratories

Recent advances in the scope and technique of hydraulic laboratories were discussed in detail by Prof. F. M. Dawson, University of Wisconsin. There is a definite trend away from the ascertainment of over-all coefficients to the determinations of refinements, including fluids other than water. In answering the question as to the function of a hydraulic laboratory, he said that for undergraduates the purpose is primarily one of illustration and demonstration. Outside engineers should also be served, in his opinion, to acquaint them with essential technical advances. Tradespeople should have an opportunity to present their problems, and the state should pass down hydraulic information to such trades as plumbers, steamfitters and boiler makers. This would not require much equipment but would take a large amount of time and ingenuity.

No major structure involving hydraulics, Prof. Dawson emphasized, should be built without the testing of one or more models. For fundamental advances and refinements, laboratory workers must correlate hydraulics with the latest developments in the field of physics, chemistry and mathematics. The microscope, high-speed movie cameras and supersensitive thermometers and recording apparatus should not be overlooked. He predicted that ultimately many laboratory-model experiments may use a "standard" liquid other than water, lighter and less viscous; from the results obtained the effect of water on larger structures might be forecast.

Studies of silt problems

In a scholarly résumé of the laboratory treatment of silt problems, L. G. Straub, associate professor of hydraulics, University of Minnesota, extended the subject to include correlation with field observations. One of the important conclusions was that investigations have advanced to the point where no necessity exists for gross mistakes in the design of structures involving silt problems, as there has been in the past. Laboratory methods, however, will not give accurate quantitative results except by proper correlation with field studies. In common with all laboratory studies using models, the investigation of sedimentation and erosion should consider the principles of mechanical similitude, including definite assurance that the motion occurrences in the model are similar to those in the prototype. Geometrical similarity does not suffice.

F. T. Mavis, associate professor, University of Iowa, in charge of the hydraulic laboratory since the death of the late Prof. F. A. Nagler, spoke on laboratory treatment of hydraulics, confining the discussion to problems under steady flow conditions: (1) hydraulics of river structures, including dams, and spillways, locks, sluiceways, diversions and cofferdams; (2) hydraulics of fixed-bed and erodible or shifting-bed reaches.

One of the problems discussed was that of university laboratories setting up a policy relative to doing work for private individuals or corporations. The demarcation between what is in the public interest and what work clearly belongs to the private laboratories was fairly well defined. Monroe L. Patzig, an operator of a private laboratory, held that it was not fair for the universities to equip their graduates to do laboratory work and then compete with them. When work is forced onto the university laboratory by pressure of various kinds, he said, the private operator would have no complaint if the "going" charge were made. For highly specialized investigations of general value, he saw a legitimate field for the state-endowed laboratories if the results were made public. With these views Dr. Willard and a number of others were more or less in accord. Prospective codes regulating such practices came in for brief notation.

The inspection of the University of Iowa laboratories disclosed numerous hydraulics experiments under way including: a glass flume 30 in. wide used for testing the model of a tainter gate, spillway and stilling pool for the Mississippi River dam at Alma, Wis.; a model of a 7-mile reach of the Mississippi near Alton, Ill., on a scale of 1:400 horizontal, 1:60 vertical, with Dam 26 and a sand-bed model; a model of the Mississippi dam at Alton in a 16-ft. channel, 300 ft. long; a fixed-bed model of the Mississippi near Fountain City, Wis., and another fixed-bed model of a 5-mile reach at Alma. Research projects are being carried on in cooperation with the U. S. Bureau of Agricultural Engineering, the U. S. Engineer office and the U. S. Geological Survey.

CWA Paves Street-Car Tracks in Seattle With Precast Slabs

THE PAVING of nine miles of double-track street-railway lines in Seattle, Wash., with precast concrete slabs developed into the largest CWA project carried out in the state. In addition to providing work for several hundred of the city's skilled and unskilled laborers, the track-paving project will eliminate a serious traffic hazard. The city has a number of miles of unpaved street-railway tracks that have been the cause of frequent motor accidents and several deaths during recent years.

Precast reinforced-concrete paving slabs were first placed in an experimental section of track in 1927, and several additional blocks of track were paved in 1931. One of these sections was 8,000 ft. long and served for several months as the approach to the Aurora Ave. Bridge, which carried as many as 25,000 vehicles per day. These slabs are 4 in. thick, heavily reinforced, and are provided with lifting holes to permit easy removal for track repairs by truck-mounted derricks. The slabs are laid directly on the ties. After seven years of continuous service the first units laid are still in good condition.

At the start of the CWA program last fall, Seattle officials made extensive studies to determine projects that would employ large numbers of men and at the same time provide needed permanent improvements. Public demand for street-car track pavement and the inability of the municipal railway to finance the work indicated this project as a possible use of CWA funds. Federal approval was promptly granted, and an organization was set up under the direction of A. M. Young, public works engineer, who represented both the city and the CWA.

Manufacture of the slabs, which comprises a major part of the job, has been carried on at three improvised plants. The main plant was set up in the Seattle warehouse where 60,000 sq.ft. of floor space was covered with forms for casting slabs. Ready-mixed concrete was delivered to the casting plant and transferred to buggies. These moved over plank runways, placing concrete in the forms covering the floor. Concrete was then spread, tamped, spaded and finished. The following day the slabs had set enough to strip the forms and remove the slabs for curing. Trucks are used to transport the cured slabs to track locations.

More than 67,000 slabs, weighing about 500 lb. each, are required by the contemplated project. Manufacture involves 910 tons of reinforcing steel, 4,600 cu.yd. of sand, 6,900 cu.yd. of gravel, 14,400 bbl. of cement, 375,000 bd.ft. of lumber and other materials. The work, therefore, spreads the benefits beyond the direct employment given to Seattle workers, since most of the materials used are the result of labor.

The project has been carried out by regular laborers paid by CWA under the supervision of R. R. Hubbard, superintendent of construction for the Seattle engineering department, and M. O. Sylliaasen, city engineer.

MAIN CASTING PLANT (above) for making precast concrete slabs for paving street and railway track. About 3,800 slabs were cast daily in this old warehouse building. Men in the background are removing forms from previous day's castings. Concrete track slabs in place (below) on double-track line. Elliptical holes are provided for insertion of lifting tong to remove slabs for subsequent track work.

Book Reviews and Notes

A Monthly Commentary on Current Additions to the Civil Engineer's Library

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A Colossal Undertaking

By Abel Wolman
Chief Engineer, State Department of Health, Baltimore, Md.

PUBLIC SEWERAGE SYSTEMS IN GERMANY—By J. Brix, Karl Imhoff and R. Weldert. Two volumes, pp. 1,549 text, 298 figures. Published by Gustav Fisher, Jena, Germany. $56.

INTERESTED OFFICIALS in this country have frequently expressed the wish that some remote but competent group of individuals, adequately endowed with technical and financial facilities, would prepare a comprehensive and authoritative compendium of the sewerage systems and sewage-treatment plants in the United States. The mere mention of this task has resulted usually in frightening off most efforts, owing to the obviously vast amount of work involved in the collection of the data, in the compilation of the facts and in the costly publication of the findings.

While the task is under debate in this country, these two volumes from Germany give concrete demonstration of the fact that it can be done and, when done, is of inestimable value.

The work here reviewed represents the completion of a stupendous undertaking. The detailed information on the sewerage facilities in all of the German cities, arranged alphabetically, is given in Vol. I. The degree of detail, of course, varies with the size and complexity of the system, but in every instance brief descriptions of the sewage-treatment plant, of pumping stations, of the basis for design and of the costs are included. In many instances, maps and detailed drawings are presented.

Volume II contains, among other things, the general principles underlying the design of public sewerage systems and of sewage-treatment plants, data on water consumption for domestic and industrial uses, details of the laboratory examination of sewage and sludge and descriptions of administrative units for the control of sewerage systems and sewage treatment, such as sanitary districts.

An unbelievable number of other problems involved in sewerage and sewage treatment appears in this same volume, such as detailed data on the populations of all the communities under discussion, on the central control over sewerage programs, on water consumption in German cities, lists of important patents and a detailed bibliography on the general subject.

It is difficult, in a brief review, to do complete justice to the enormous amount of work involved in the preparation of these two volumes. They are packed with interesting, general and detailed information, which should be available to practitioners. The text supplies a new reference volume that we might well emulate for similar facts in this country. The reviewer shudders at the task of compiling such a text, but its value when completed is obvious.

The books are excellently printed, the paper is of fine quality, the drawings and photographs are carefully reproduced, and an obvious and successful effort has been made to provide the reader with easy reference to all of the information. The authors are to be congratulated on the accomplishment.

A New Approach to Earthquake Design

DYNAMICS OF EARTHQUAKE-RESISTANT STRUCTURES—By Jacob J. Creskoff. Cloth; 6x9 in.; pp. 127; tables and diagrams. Published by The McGraw-Hill Book Co., New York and London. $2.50.

IN STRUCTURAL DESIGN, earthquake and wind loads have always been assumed as static, with full knowledge that the assumption was wrong but with the firm belief that an attempt to utilize true dynamical assumptions was not only difficult but uncertain and that static designs had in general proved satisfactory in service. The author of this book started out with the idea that only a dynamical design was permissible for earthquake loadings and that if such a design could not be made to fit conventional buildings, the buildings should be altered as necessary. He has produced a monograph on a design procedure that is notable alike for its brevity and mathematical simplicity. Some of his basic assumptions can and have been challenged, but he maintains that the errors, if any, are small and that some of the misconceptions arise from thinking in terms of slender steel skeletons with thin curtain walls and floors instead of in terms of a building that is a rigid unit. Without question his approach is unique, the reasoning is stimulating, and for the most part patently logical.

After several introductory chapters dealing with earthquake history, seismography and period and amplitude of particles subjected to earthquake waves, Mr. Creskoff turns to the dynamics of earthquake motion. First developing equations for period and amplitude for free and forced vibrations with and without friction, he then applies these to various types of beams—that is, beams with various degrees of end fixity. Equations are deduced for both the fundamental period and the first harmonic period of the various beams, and a notable tabulation of coefficients is the result (p. 48). This tabulation shows, for example, that the fundamental period for a certain type of beam is 1.6 times as large, and the first harmonic period is 1.4 times as large as for another type of beam. With this knowledge, the period of a given beam may be altered (by changing its end conditions) until there is no danger of synchronization with the dangerous bracket of earthquake periods, assumed by the author to be 1 to 1.5 sec.

The development briefly sketched above establishes the fundamentals of the design method. In applying the data to building design, which is largely a matter of distributing the seismic shear and moments, the assumptions are made that the mass of the structure is concentrated at the floor levels, that the floors deflect as units, and that the ends of the columns and walls are fixed. In general, exterior walls only are considered as resisting elements, and reinforced-concrete walls are indicated. Equations are developed for the necessary distribution of shear and moment.

The last three chapters are devoted respectively to "Application to Buildings," "Aseismic Design of a Tall Building" and "Aseismic Design of a Short Building." The book is a challenge to current methods of design.

A Contractor's Idea of Safety

SAFETY AND ECONOMY IN HEAVY CONSTRUCTION—By Ray J. Reigeluth. Fabrikoid; 5x8 in.; pp. 237; photographs, tables and line drawings. Published by McGraw-Hill Book Co. Inc., New York and London. $2.50.

Too many books on construction safety are filled with theoretical ideas of how to operate an accident-free job, combined with a formidable list of "don'ts" and "thou shalt nots." Here is a book full of practical ideas on safety, written by a member of an old and experienced contracting firm. Individual chapters are devoted to a discussion of safe practices in the use of the common tools of the contractor, such as rigging, boilers, cranes and derricks, piledrivers, explosives and shoring. In a logical manner the author ties up safety in construction with economy. That the safe method is usually the most economical method is clearly demonstrated. Mr. Reigeluth

has placed safety in construction on a simple, sane and practical basis. From a mass of available information he has picked out the essentials and presented them in convenient form.

Miscellaneous Notes on Booklet and Reprints

THE AMOUNT AND CHARACTER OF TAXES paid by railroads is the subject of a pamphlet entitled "Railway Taxation" published by the Bureau of Railway Economics (an agency supported by the railroads), Washington, D. C.

A REVIEW of modern conventional suspension-bridge design practice is contained in a 100-page booklet written by Carl M. Bohny and published by Wilhelm Ernst & Son, Berlin, Germany. It is entitled "Hangebrucken Beitrage zu ihrer Berechnung und Konstruktion."

A NEW EDITION of the 450-page Handbook of Arc-Welding Design and Practice has recently been published by the Lincoln Electric Co., Cleveland, Ohio. New information is included in the second edition on weld inspection, stress-distribution studies by photoelastic method, procedure and speeds for welding aluminum, technique for welding copper and construction of welded water lines. Price, $1.50; foreign, $2.

THE REPORT ON BUILDING RESEARCH of the British Building Research Board for the year 1932, available from His Majesty's Stationery Office, London (2s.6d.), contains some 125 pages devoted to the general research and special investigations of that body. The report contains a summary of the cement and concrete research that is believed to be as complete as that carried on anywhere in the world. Studies of nearly every building material except steel are included.

GUIDE TO GRADING STRUCTURAL TIMBER, by T. R. C. Wilson of the Forest Products Laboratory, gives a basis for comparing structural lumber grades definitely on the strength principle. Issued as Miscellaneous Publication 185 of the U. S. Dept. of Agriculture, the pamphlet presents the factors that affect strength, shows how to define and limit them in grade specifications and how to determine working stresses for any given grade in a wide range of commercial species. 5c. from Superintendent of Documents, Government Printing Office, Washington, D. C.

THE STRENGTH of thin cylindrical steel shells and columns is the subject of an investigation made at the University of Illinois and published as Bulletin No. 255 of the Engineering Experiment Station, Urbana, Ill. (Price 50c.). The object of the investigation, which was conducted in cooperation with the Chicago Bridge & Iron Works, was to study the type of failure characterized by local wrinkling and to investigate the dependence of the wrinkling strength upon physical properties of the material and the geometrical properties of the cylinder, including end conditions. Columns of this type have been used to support elevated tanks, notably one at Tallahassee, Fla. (*ENR*, March 30, 1933, p. 403), and such columns are simulated by the riser pipes in all conical, elliptical and radial-cone-bottom elevated tanks.

New Books and Revised Editions

[Those desiring copies of the books listed below or mentioned elsewhere in this section should order them from the publishers or from their local booksellers.]

BUILDING AN ENGINEERING CAREER—By C. C. Williams. Cloth; 6x9 in.; pp. 247; numerous diagrams and half-tone illustrations. Published by McGraw-Hill Book Co., Inc., New York and London. Price $2.

CENTRIFUGAL PUMPS, TURBINES, AND PROPELLERS—By Wilhelm Spannhake. Translated from the original German by J. B. Drisco. Cloth; 6x9 in.; pp. 328; numerous diagrams and half-tone illustrations. Published by The Technology Press, Cambridge, Mass. Price $5.

Letters to the Editor

Errors in Two Recent Articles

Sir—In the article by J. R. Burkey in your March 29 issue, p. 401, entitled "Architectural Detail Emphasized in Bascule-Span Bridge," three errors occurred. The contract price was quoted as $575,825, whereas it should have been $175,825. W. S. Hindman acted as coordinating engineer in the design of the structure and had direct charge of the field work, while the architectural design of the control houses was handled by Josephine Power. These names were not given correctly in the article.

Columbus, Ohio, April 4, 1934. J. R. BURKEY, Chief Engineer of Bridges, State Dept. of Highways.

Sir—In reference to my article on lake-level control by submerged sills (*ENR*, April 5, 1934), there is an error on p. 438 in the second paragraph of the middle column. The words "within the limits of 350 to 700 ft." should be omitted, the sentence in question reading: "For the channel without sills, the velocities were unevenly distributed, but with the sills became much more uniform." W. F. HEAVEY,

Cleveland, Ohio, April 7, 1934. Major, Corps of Engineers.

Salaries for Engineers

Sir—In your issue of March 15, p. 359, there appears an article entitled "Prevailing Salaries Outlined by Civil Engineers Society." This article gives a schedule of key salaries determined from a survey conducted in 1930 and "the appropriate reduced salaries as of March 1, 1934."

In the first paragraph of the article it is stated that the report is offered only as an aid to those who are working on wage schedules for the engineers' code. Yet the words "appropriate" and "justify" appear several times, leading quite naturally to the supposition that this wage schedule represents the amounts engineers should be paid. The wording of the final paragraph of the article leaves little doubt that the base schedule given is suggested as the proper one. It is ironical that the information used was originally gathered by the society's committee with "the final aim of finding measures which can be employed in raising the various salaries to amounts which seem commensurate with the services the engineering profession is rendering to the public," according to *Civil Engineering* for March, 1932.

No one can dispute the facts with the Am.Soc.C.E. committee that made a compilation of salaries paid in 1930. However, the recommended figures for March 1, 1934, have apparently been taken out of thin air. The committee on salaries was appointed in 1927, by 1930 had questionnaires distributed, and by 1932 had analyzed the results. It is hardly credible that in two months reliable figures for 1934 can have been gathered, considering the length of time necessary to obtain the figures for 1930. If, on the other hand, the 1934 schedule is offered as "appropriate," as indeed your article states, it does not merit one moment's consideration, since prices in most lines are back up to 1930 levels and getting higher, so far as they affect living costs.

Helena, Mont., March 20, 1934. WILLIAM BERTWELL.

THE RECENT supplemental report to the committee on salaries of the Am.Soc.C.E. was prepared and published in response to an urgent request for concrete facts as to *actual salaries paid* as opposed to abstract theories as to what engineers ought to be paid. This limitation must be taken into consideration in studying the report. Under the PWA and CWA, organized labor is paid, generally, on the basis of the "prevailing wage" for each class in the particular area. Engineers, frequently, were being paid rates little above that of common labor, and those who oppose such low pay are handicapped for lack of any authoritative information as to what wages actually were being paid to a sufficiently large number of engineers to be termed "prevailing rates." The salary committee statement supplied that information. The term "appropriate" as used in the statement is not, therefore, used in the sense that Mr. Bertwell interprets it as being used. —EDITOR.

ENGINEERING NEWS-RECORD

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A Semicentennial

SOME TIME this spring marks the fiftieth anniversary of the completion of the Home Insurance Building in Chicago, generally conceded to have been the first building actually to utilize a skeleton frame with curtain walls—the distinguishing characteristic of the modern skyscraper. High land value was the motivating force in the genesis of the skyscraper, as it has been in much of its subsequent development. This development has been unique in speed and accomplishment, from the first cautious groping toward thinner walls, smaller foundation piers and increased height to the present confident use of tower structures limited (and more recently questioned) chiefly on the score of economic considerations. The development is unique in another respect. On its 45th anniversary the skyscraper was the pride of the masses, of real estate promoters, of architects and engineers alike. Now, only five years later, it is being subjected to all manner of abuse. The economic forces which brought it into being and fostered its development have deserted it. Whether or not any more thousand-foot buildings will be built can hardly be predicted; the chances are as good one way as the other. It is a certainty that many more buildings of skyscraper type will and must be built in the cities of this country. And such buildings must be more efficient, more economical to construct and operate than their predecessors. On its semi-centennial the skyscraper gives to architects and engineers reason for satisfaction in a job well done and promise for a field of activity calling for ever-increasing ingenuity and skill.

Not As Black As Pictured

ENGINEERS will be surprised to learn that "The majority of engineers are prompted primarily by self-interest." This statement is made by Paul H. Norgren in an article entitled "The Overrated Engineer," published in the April issue of *North American Review*. Not only are engineers interested chiefly in gain, they also lack scientific methods of approach to their problems—they use cut and try methods, thus reaching a solution by mere chance and at considerable waste of materials and time! Thus one engineer rates his fellows; and his fellows might have cause to regret the publication of this characterization in a magazine written for thoughtful people were it not for the fact that the rating is so grossly faulty that it is not likely to mislead anyone. In fact, the same thoughtful readers, looking at the engineers of their acquaintance, will find little or no evidence that their chief object in life is material gain; and looking at the engineering structures around them, they will find little evidence that these were designed without regard for the scientific data available to the designers. All about us we have machines and structures still giving efficient service after years of use, machines and structures that would have been forced into discard by economic conditions had they not been skillfully designed and well built. Engineers are not supermen, nor do they claim to be; but they have, we believe, done as well as any others in advancing human progress, and they have done it with an amount of self-sacrifice equal to that of any other profession or craft.

The Worm Turns

DEFEAT of the one-cent emergency gasoline tax by the New York legislature—or its near defeat, whichever way the last-minute political bargaining may turn—is an advance sign of general revolt against the exploitation of road users by diversion of road revenues from their designed purpose. Nearly all the states, after once trying such diversion for a momentary emergency of income shortage, found it so easy a device for procuring money that they continued and increased it, using the funds so obtained for every conceivable purpose, but in ultimate effect and intent for reduction of property taxes. Organized real estate interests, loud in protest against the tax burden, awed the legislators into shifting part of their burden to others; the road users, taken by surprise, were unready to act and remained silent. In consequence of the diversions of road funds from their proper field, road construction and maintenance workers were thrown out of employment in large numbers, to be "relieved" by charity distribution of the very money that should have kept them in useful employment for the public benefit. Mass sentiment against the inequity of this procedure has gathered great strength in the meantime, and, when the state authorities of New York undertook to disregard their promises of a year ago that the emergency tax would be limited to one year, the opposition became articulate. Whether or not it wins in the present contest, the campaign against class exploitation of road users is certain to continue until the present injustices are ended.

Twenty Years' Labor

TWENTY YEARS AGO this spring there was begun a joint study in stresses in track that has developed into one of the most comprehensive and thorough investigations of an engineering nature ever undertaken in this country. From it has come much new knowledge of stress distribution in rails, in rail joints, in ballast and in the roadbed, and many subsequent developments in the design of these elements of track structure as well as in the design of locomotives owe their origin to conclusions drawn from the results of the work of the track stress committee. This twentieth anniversary should not pass without a word of acknowledgment to those who have been responsible for these fruitful results. Chief among them is Arthur N. Talbot, professor emeritus at the College of Engineering of the University of Illinois, who first saw the need for such a study, who laid down the plan upon which it was begun, and who has served continuously as chairman of the committee which represents the American Railway Engineering Association and the American Society of Civil Engineers. And back of Professor Talbot have stood many other men, largely members of both societies who have made up the committee, as well as a group of chief engineers of railroads who have participated in the field work and a technical staff that has carried on the field and office work. Support for the extensive field tests has come principally from the American Railway Association and the steel companies specializing in the manufacture of rails. The work has been carried forward in an orderly fashion, highly ingenious instruments have been developed to

make possible the field work, and, progressively, the conclusions have been published in the committee reports so that they could be put to practical use at the earliest moment. The study is an enduring monument to all who had part in it, and the civil engineering profession as a whole owes them a debt for their twenty years' labor.

Vibrated Pavement Tests

IMPORTANT RESEARCH RESULTS on the efficiency and economy of compacting concrete pavements by vibration are given in a group of articles beginning in this issue. The experimentation so far conducted, while in a measure tentative, leads to the definite conclusion that vibration is an entirely practicable process of consolidation, when properly directed, even under the disadvantage which prevails in paving—namely, that the vibratory action can be applied to one face of the concrete mass only. With the practicability of the process thus established, the tests described show further that vibrated pavement has greater strength and density than concrete pavement laid without vibration, and that it costs materially less in cement requirement. On the other hand, vibration gains these advantages by using harsh mixtures, and the latter impose certain construction and equipment requirements that may not be neglected if full success is to be attained. Vibration, in short, does not consist merely of an added operation; it introduces a concrete of different workability and additional stresses to be resisted by road forms and subgrade.

A Simpler View

THERE Is a definite gain in dealing with complex analyses and calculations where it is possible to obtain a clear and simple view of the relations which they involve. It is in giving such a view, a simpler one than has been available to engineers heretofore, that the method for analyzing indeterminate structures which Professor Ellis presents this week under the name of the Williot strain method is particularly interesting. The method is an aid to calculation, but this fact is perhaps less important than the fact that it gives a simple understanding of the structure's behavior. To most engineers the analysis of indeterminate or hyperstatic structures is a mere maze of algebraic operations, whose rational purposes is wholly befogged by its mechanism. Actually there are quite simple geometrical relations between the distortions of the several members of a frame, and by combining these relations, which yield one equation for each element of the frame containing a redundant member, the final operation becomes a mere matter of solving simultaneous equations. It is this view that is made available by Professor Ellis' method.

A Retrospect and A Look Forward

SIXTY YEARS AGO this month *Engineering News-Record* first saw the light of day, when George H. Frost, a Chicago surveyor, brought out a small monthly called *Engineer and Surveyor.* That beginning later grew into the weekly journal *Engineering News;* still later this was joined with a similar enterprise founded by Henry C. Meyer in 1877, *The Engineering Record.* From these elements present paper has grown.

As we reach this birthday anniversary we think back to the succession of distinguished men, the publishers and editors, whose energies guided the paper's steps through the multitude of significant events that mark the past six decades of engineering progress. Even more strongly there crowd in upon the mind recollections of the great engineers who in that long period carried the art forward to achievements and possibilities undreamed of in the seventies.

In 1874 engineering was a unit, something far different from the great group of intensively specialized arts that bear the name of engineering today. Throughout the marvelous evolution *Engineering News-Record* has at all times sought to serve fully the interests of the civil engineer as one who plays a determining part in shaping the world's physical form. It has endeavored to portray and record the advance of engineering, to set forth the best thought of its ever-growing number of votaries, to guide and lead as far as might be toward the newer and greater responsibilities of the morrow. The development of water purification, of reinforced-concrete construction, of the bridge engineer's art, of buildings and foundations, were notable parts of its concern. And throughout, also, the editors depended on the profession's cooperation. That this cooperation was given in most generous measure is one of the high satisfactions that the work of the paper has brought to all connected with it. Upon its continuance we rest our hopes for future useful service.

How vast has been the change in the engineer's existence during the past sixty years needs no restatement. Now, more than ever, should these changes be kept in mind, when we see the profession in the grip of great centrifugal forces, when there are apprehensions that the profession as such may vanish and give way to a mere group of business activities. The same profession which sixty years ago knew nothing of the impressive engineering staff organizations that today carry out the work of railroads, industries, cities and states has seen its objectives change as greatly as its organization and methods. It is now interlinked with economic activities, with extensive industries, with the whole chain of social purposes and movements that determine the country's development.

As we look forward, the certain prospect of change as great as that of the past appears before us. The engineer will be called into new service, to exercise larger functions and meet heavier responsibilities. He will continue, we may be sure, the transformation from the status of technical agent of applied science to the status of an active unit in industry, in administration and government. He will remain the engineer at all times, however, and carry into each new field his method of working with predetermined forces from weighed and measured facts. He will often have to calculate and construct in terms of economic quantity rather than feet or pounds, but his attitude of mind as well as his processes will remain those of the engineer.

It is along this path of advance that the profession's course will lie. Its newer mission is to help in arranging the complex puzzle of modern community existence in better order, to make life more efficient, more secure and more satisfying to all. By bringing to bear on this task the powers of objective engineering method, the profession of the future will be an incomparably larger factor in the world's concerns than was thot of the past. To the extent of its ability, *Engineering News-Record* will endeavor to assist in realizing this end, and with the profession's continued good will and cooperation it looks forward to a long period of further service.

CURRENT NEWS

Three Divisional Code Authorities Start Functioning

THE ESTABLISHMENT of the first three divisional code authorities has been announced and the approved chapters of the construction code and national headquarters are:

Chapter III—National Code Authority for Painting, Paperhanging and Decorating Division, 701 Brown Building, Philadelphia, Pa.; John B. Dewar, chairman; G. S. Stuart, executive secretary.

Chapter IV—Divisional Code Authority for Elevator Manufacturing Industry, Inc., Room 306, 101 Park Ave., New York City, John McArdle, administrator.

Chapter IX—Tile Contracting Code Authority, 600 Investment Bldg., Washington, D. C., H. R. Cole, executive officer.

First area hearing

Announcement has been made by NRA of a public hearing, May 1, in Washington, D. C., in connection with application for approval of a proposed mutual or area agreement, affecting the mason contractors in New York City and part of Long Island. This is the first area agreement submitted for approval under Article III, Section 1, Chapter I of the Construction Code, and Section 7 (b) of NIRA. If approved by the President, this agreement would fix the standards of hours of labor and rates of pay for the territory described, which would be enforceable upon all employers, whether or not they were signatories to the agreement, by charging them with unfair competition under the code. A special joint employer-employee board would be forthwith appointed to investigate and handle complaints of non-conformance with the agreement.

The agreement is to run until February 28, 1936, and was made by the Mason Builders' Association of Greater New York, and certain bricklayers unions. It fixes the wages for bricklayers at $1.50 per hour, gives a five (5) day week and maximum of forty (40) hours per week with the hours of labor from 8 a.m. to 5 p.m. All work between 8 a.m. on Saturday and 8 a.m. on Monday, and between the hours of 5 p.m. and 8 a.m. on other days, is to be considered as overtime which shall be compensated at double rate; but shall not be paid to persons working on the second or third shifts. However, in cases of the second or third shift, eights hours pay is granted for six hours work.

The construction code authority has issued six explanations of the Construction Code, based on specific conditions formally presented for such explanations.

In explanation No. 1, the authority states that the architect is not an awarding authority within the meaning of the code when he merely takes bids and submits them to the owner for awarding. In this case, the owner is the awarding authority.

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New Jersey Imposes Code on Steel Fabricators and Erectors

Among the first codes to be signed in New Jersey, following enactment of legislation setting up a state organization patterned after the NRA, was one for structural steel and iron fabrication and erection. All of its provisions are not yet known, but a code authority of seven members is provided for, two to be appointed by the governor. Minimum wages are to be set by the authority, which may vary in different parts of the state, and a maximum 40-hour week is decreed. It is understood that minimum wages are higher than NRA code wages. Whether an attempt is contemplated to impose state code wages on companies fabricating elsewhere for erection in New Jersey cannot be now ascertained. Published copies of the code, which will be administered by the "Structural Steel and Ornamental Iron Association of New Jersey, Inc.," will be available from 1060 Broad St., Newark, next week. Governor Moore has appointed as the two independent members of the code authority John J. O'Neill, business representative of the Ironworkers' Union in Hudson, Bergen and Passaic Counties, and Hugh A. Kelly, Jersey City.

Hillside Housing Project in New York Started

Ground breaking ceremonies on April 19 marked the start of construction on the Hillside Housing Corp. project in the Bronx, New York City, known as Hillside Homes. The contractor's buildings are completed, some shovels are at work and by May 1 the job will be in full swing.

Hillside Homes is the second PWA-financed limited-dividend project to get under way, Boulevard Gardens in Queens, New York City, being the first. Located on the Boston Post Road at 214th St., on a 712,000 sq. ft. site not previously occupied by buildings, it will provide housing for some 1300 families in 4-story walk-up apartments and a few 6-story elevator buildings. Fireproof construction is to be used—brick walls and concrete floors. Ground coverage is 29 per cent, with gardens, playgrounds and wading pools occupying the remainder. One street cuts through the site and pedestrian underpasses are provided. Rents will be about $11 per room. The total cost is $6,100,000 of which the owners equity is $900,000. Clarence Stein is the architect and Starrett Brothers & Eken, the builders.

Illinois Waterworks Problems Discussed at AWWA Section Meeting

DISTRIBUTION PROBLEMS, accounting, plumbing and its responsibilities and plant descriptions made up the program of the spring convention of the Illinois Section of the American Water Works Association, April 18 and 19 at Peoria, attended by about 150. A resolution of protest was directed toward the dilatoriness of PWA in getting allotted money to work on meritorious projects as well as requesting a more liberal policy in the acceptance of collateral. Statements on the disappointly slow procedure and failure to accomplish the re-employment intended by the National Recovery Act were combined with a request for additional funds to cover the good non-federal projects among the approved applications in Washington. Most criticism was directed at the meticulousness of PWA employees, particularly those passing on collateral.

The coupling of a water department's credit with tax delinquency, particularly special assessment delinquency, as bearing on the merit of water revenue bonds seemed especially irksome and nonsensical to many. Generally speaking, in cloakroom discussion, the PWA came in for decided condemnation and the often-repeated assertion that the return of former days of normal financing with investment houses would be more than welcomed.

As at the Indiana Section meeting the previous week, a definite move was made looking toward the better and safer supervision of waterworks operation through licensing of one or more officials in a department or company responsible for the safety and sanitary quality of the water supply. A legislative committee was authorized with general instructions to cooperate with the state board of health and other organizations interested. Discussion brought out the futility of attempting to stress tenure of office and the evil resulting from the removal of numerous officials in political upheavals. In Indiana the displacement of more than 50 waterworks officials following the spring elections has given the state board of health much concern. Figures for Illinois were not given but displacements were considered unduly high.

Rehabilitation of the Cicero Water Department which buys water from Chicago through three connections and then boosts it to a fully metered system was described in detail by Frank J. Broz. (*ENR*, Mar. 29, p. 408.) Stopping leaks and correction of meter underregistration and irregularities in meter reading and billing turned deficits into profits. CWA engineers were used to bring records up to date. Six projects employed 215 men who painted hydrants, constructed valve boxes and made a leak survey. Sixty men in one 48-block area found 791 buildings with one or more leaks in the water pipes. Under city employee direction 100 men reset meter boxes to make them accessible.

Herbert Schmitt, Milwaukee, described experiments made on lead and other joint-

(Continued on p. 553)

Money is Ready For 2,000 Non-Federal PWA Projects

CONTRACT PAPERS for the purchase of bonds and for the consummation of PWA grants have been forwarded to more than 2,000 of the 2,345 non-federal projects, according to an announcement from Administrator Ickes. The actual start on these projects will be determined by how rapidly the states, municipalities, or other public units are able or willing to conclude these agreements. It has been estimated that these 2,000 projects will provide 45,500,000 man-days of direct labor.

The PWA is now ready to advance funds for loans and grants called for by these agreements as soon as money is required to pay for work done, provided that the applicants have executed the contracts in accordance with the terms and conditions required. Many of these necessitate local legislation or bond elections. The legal staff of the PWA is now preparing the agreements for the remaining 345 non-federal projects which are principally allotments or reallotments made during the past few weeks.

Of the 2,000 contracts sent out, applicants have executed and returned 1,500 and of this number 1,150 have been signed. The remaining 350 will be signed as soon as they have been approved legally.

Eleven PWA allotments totalling $1,087,800 for non-federal projects have been rescinded and the funds already reallotted to nine non-federal projects which have been on the waiting list. The largest of the rescinded allotments was one for $500,000 to the city of Hartford, Conn., for street paving.

At the request of applicants, 28 loan and grant allotments made for non-federal projects have been changed to grants for 30 per cent. The difference between the 28 loan and grants and the grants has released $2,187,405 for reallotment. The larger individual projects in this list include a $374,000 sewer project for North Adams, Mass., a $470,500 state administration building for the prison camp at Bordentown, N. J., a $359,000 state hospital building at Greystone, N. J., and a $450,000 sewage disposal plant project at Providence, R. I.

Among the new PWA loans and grants announced during the last week, the largest single award was $125,000 to Glasgow, Mont., for the construction of a hospital. The construction of this local hospital has been planned to save the government the expense of having to erect a temporary federal hospital for the nearby Fort Peck dam project to provide hospitalization facilities for this construction work.

Employment on Public Works Increases During March

The employment on public works construction projects reached a total of 305,000 during March, which is an increase of 10,000 over the employment during February, according to a report of the Department of Labor. Payrolls for the month of March totalled $16,000,000. These figures do not include employment in supervisory or clerical capacity.

TVA Negotiating on Prices With Cement Companies

Washington advices report that negotiations between the Tennessee Valley Authority and cement manufacturers in the valley are in progress, in an effort to determine fair and reasonable prices. The belief is expressed by men close to the parties that the negotiations will lead to harmonizing the views of the two sides.

Rumors that the Authority intends to build a mill to manufacture its own cement have been denied in a letter to Representative Huddleston of Alabama by L. L. Campbell, executive assistant to Arthur E. Morgan, chairman of the TVA. Constituents of the congressman had informed him of the rumors, and protested against the TVA's going into cement manufacture on the ground that this would deprive industry of its market and prevent its recovery, by establishing added competition. When these complaints were brought to Dr. Morgan's notice, Mr. Campbell, replying for him, wrote that "the Authority does not desire to manufacture cement unless in so doing it can save a very substantial sum to the government. The Authority would prefer to rely upon the skill and judgment of the cement industry."

The rumors arose, it is said, from study by the TVA of the possibilities of utilizing Nitrate Plant No. 2 at Muscle Shoals for cement production. The plant contains a considerable part of the equipment that would be required for such production and, because of its riverside location, it could load directly to barges for cheap transportation by way of the Tennessee River. On the other hand there are a dozen or more mills in the valley and the adjacent territory, whose aggregate producing capacity approaches 20 million barrels of cement per year, but whose maximum production has been well below one fourth of this amount. Since the TVA requirements are placed at about 1 million barrels per year for the next ten years (based on the enlarged construction program now being considered), ample capacity is already available to meet them.

Recent contracts awarded by the TVA for some 300,000 bbl. cover initial construction at Norris and Wheeler dams. No large additional purchases will probably be necessary for several months.

Senate Committee Holds Hearing On LaFollette Public Works Bill

Support for the $10,000,000,000 public works bill sponsored by Senator LaFollette was presented at the opening of the hearing on the measure by the committee on education and labor.

Willard Chevalier, publishing director, *Engineering News-Record*, urged speedy continuation of PWA allotments declaring that "boldness in disbursing PWA funds is the essence of economy." He also stated "we must continue to prime the pump until it catches hold and we cannot prime it with an eye dropper."

T. Semmes Walmsley, mayor of New Orleans, representing the United States conference of mayors, advocated the concentration of allotments to municipal projects, stating "it is essential that provision be made for financing municipalities so that it will be possible for them to do all they want to on public works programs."

Underwood & Underwood Photo

PREPARING A BRIDGE FOR MOVING DAY

Preliminary to the construction of a monumental concrete bridge of three 146-ft. span arches over Rock Creek Valley on Calvert St. in Washington, D. C., the existing steel viaduct will be moved some 80 ft. downstream, to serve as a bypass while the new construction is under way. May 7 has been set as moving day, and, as the view shows, the new foundations are complete and the old bridge is being jacked up and mounted on rollers. Horses and block-and-tackle will constitute the motive power. The contract permits stopping traffic for only 48 hours. The work of moving the old bridge and building the new will be done by the John W. Cowper Co. The project is under the supervision of Major John C. Gotwals, Corps of Engineers, U. S. Army, engineer commissioner, District of Columbia.

PWA Reports on Handling of Fund at Senate Request

IN A SPECIAL REPORT to the United States Senate, Administrator Ickes has announced that allotments totalling $3,298,207,098 were made up to Feb. 15, when the entire public works program was reviewed and summarized. This report was requested by a Senate resolution, approved Feb. 15, which directed the PWA to prepare a complete review and statement of organization, policy and allotments from the $3,300,000,000 fund, both approved and pending.

PWA funds, according to the report, accounted for the direct employment of 4,613,925 persons, including the CWA workers and the CCC, which were both financed through PWA allotments. This record of employment does not take into consideration anyone employed indirectly.

The financial analysis indicates that $1,100,983,131 was allocated by legislative enactment or executive order, $1,378,846,755 was allotted for federal projects including the $400,000,000 set aside for federal aid to highways and the $238,000,000 for naval construction. The balance, or $818,377,212, has been allotted to non-federal projects.

Allotments to 15,203 projects are enumerated in the report, and of this number 13,138 were federal, 2,065 non-federal. In addition to loans and grants to public bodies and private corporations, eligible under the law, the non-federal projects include those made to limited dividend corporations for the construction of low-cost housing and to railroads for rails and equipment.

With the original $3,300,000,000 fund exhausted by allocation there remain 6,106 applications for loans and grants totalling $3,423,024,774 still pending before the PWA. The report stresses the point that precautions have been taken to protect the loans made.

Relative to the administration of the PWA fund, the report states: On July 8, 1933, with the appointment of Secretary Ickes as administrator, the first undertaking was to create a functioning organization in Washington and in the field in the shortest possible time. Regional advisers, state advisory boards and state engineers were appointed to consider non-federal applications for loans and grants. At the same time a staff of technicians was set up in Washington in the fields of engineering, law and finance. The administrator and the special board for public works, appointed by the President, rushed allotments for federal projects to permit the early start of these jobs.

The report states that the number of applications received by Feb. 10, 1934, made it necessary to direct PWA state engineers to turn down subsequent applications and to complete the examination of those on hand by Feb. 28. At the same time regional advisers and members of state advisory boards were notified that their work would be discontinued at the end of February.

This report on the entire PWA program and tabulation of allotments is a 300-page printed document (Senate Document No. 167, 73d Congress, second session) and has just been released.

Cleveland Sells Securities and Pays Defaulted Bond Interest

By accepting an offer of a syndicate of local and Toledo bond houses to purchase a million dollars (face value) of city bonds for $932,000, Cleveland moved to erase its default on bond principal payments of February, March and April. This $932,000, accepted April 20, together with an additional $881,000 cash from the city's sinking fund, has been transferred to the Irving Trust Company of New York, as Cleveland's bond paying agent, to meet the overdue payments (with interest to April 3, 1934) on the defaulted bonds.

With less than $200,000 a month to divide among the contractors, Cleveland has been paying out of its own bonds the month-to-month cost of work on construction projects for which it hopes eventually to secure $13,000,000 in PWA loans and grants. Allocated to Cleveland last Autumn, and with $1,687,000 of the amount already deposited in Cleveland banks, the whole allotment was abruptly withdrawn by the Public Works Administration when the city defaulted Feb. 1 on its bond payments.

Despite the fact that the city has carried on without a federal appropriation, on a limited basis, PWA inspectors have remained at the sewage plant and the garbage incinerator, projects where work is underway and on numerous smaller water main installations.

The expected visit to Cleveland this week of H. L. Landau, chief counsel for the PWA, to discuss the city's loan and grant applications with Mayor Davis and members of his business committee, is expected to do much to clarify the situation.

Sir Frederick Palmer, British Bridge Engineer, Dies

Sir Frederick Palmer, well known British civil engineer and bridge builder, died at Lingfield, near London, age 72. He was engineer for many important bridge projects in various parts of the British Empire, worked for years on railroad construction in India, and also served as chief engineer for the Port of London Authority when it was organized in 1909.

He was born in 1862 and educated at Neath. After serving for a short time as assistant engineer on the Great Western Railway, he was appointed to a similar capacity on the East Indian Railway through the recommendation of Sir Alexander Rendel, consulting engineer to the government of India, whose partner he became nearly 30 years afterward. He remained with that railroad for nearly 18 years and in 1901 became Calcutta Port engineer, holding that position for 8 years and returned to England in 1909 as chief engineer to the newly constituted Port of London Authority.

In 1925 he was appointed consulting engineer in charge of extensive work for the Port of London Authority, and in 1927 and 1928 acted as consultant for the development of several ports in Canada.

In 1929 Palmer was named engineer for the proposed Charing Cross bridge, London, estimated to cost $62,000,000.

He served as president of the Institution of Civil Engineers in 1926-27; he was a member of the Am. Soc. C. E.

Major Gen. Wm. C. Langfitt, Chief Engineer of A.E.F., Dies

Major Gen. Wm. C. Langfitt, chief engineer of the American Expeditionary Force in France from July, 1918, until July, 1919, died at the Walter Reed hospital in Washington on April 20, age 73 years. Gen. Langfitt retired from the army in 1920 at his own request in order to enter into private business. Subsequently, in 1926 he returned to government service for a time to serve as a member of the board appointed to pass upon the plans for Coolidge dam.

General Langfitt was born in Wellsburg, W. Va., in 1860, graduated from the U. S. Military Academy in 1883, and for the next 15 years was on river and harbor work, following the normal course of a young army engineer officer. In 1898 he was appointed to the command of the 3d Battalion, Second Volunteer Engineers, and assigned to the Philippine Expeditionary Forces; subsequently was placed in command of all the U. S. forces in the Hawaiian Islands. Returning to this country in 1899 he was again in river and harbor work and then was for a time instructor at the engineer school, Washington Barracks, succeeding to the command in 1907 and continuing in command until 1910.

With the outbreak of the World War Colonel Langfitt was placed in command of the 13th Engineers (Railway) and went to France in command of that regiment. Soon after reaching France he was promoted to the rank of brigadier-general and placed in command of the U. S. forces serving with the British Expeditionary Forces. In this capacity he spent much of his time on the active sections of the British front. Subsequently General Langfitt was made manager of light railways, serving in that capacity from September, 1917, to March, 1918, and then was made chief of utilities, which position he held until his appointment to be chief engineer, A.E.F., in July, 1918. This appointment was made by General Pershing in recognition of his demonstrated ability as an organizer and executive.

Upon his return to this country following the war General Langfitt served as division engineer for the Northeast Division at New York until his voluntary retirement in May, 1920, following the President's failure to name him Chief of Engineers, U. S. Army.

Milwaukee Appeals Activated Sludge Case to Supreme Court

Following the refusal of the U. S. Circuit Court of Appeals to give Milwaukee a rehearing on the activated sludge patent infringement suit, the city authorities announce that an appeal will be taken to the U. S. Supreme Court. On February 7, 1933, Judge F. A. Geiger, of the United States District Court held that the patents on the activated-sludge process were valid and had been infringed by Milwaukee. The Circuit Court of Appeals upheld the District Court's decision on March 2, 1934. The city's appeal to the Supreme Court estops Activated Sludge, Inc., the owner of the patents involved, from proceeding to collect damages and royalties pending the outcome of the appeal. More than 100 municipalities are interested in the ultimate outcome of the Milwaukee case.

Railroad Tunnel 11½ Miles Long Is Officially Opened in Italy

The longest double-track railway tunnel in the world piercing the Italian Apennines with a 11½-mile bore was officially dedicated by Premier Mussolini and King Emmanuel on April 22. The new railroad route between Florence and Bologna, Italy, made possible by this tunnel, cuts the distance from 80 to 60 miles and the train time between Rome and Milan is cut from 10 to 8 hours.

Work was started on the tunnel about 20 years ago, and after serious delays resulting from the war and its after effects, the project was finally completed by the Fascist régime. The cost of the tunnel alone is estimated at more than $40,000,000, and the general method of driving was outlined in *Engineering News-Record* July 9, 1931, p. 65. At the opening ceremonies, the King dedicated a memorial fountain honoring the 98 men who lost their lives during the construction work.

The railroad system will be modernized to include automatic signal control, electrified operation and other improvements making possible train operation at 80 miles per hour. The tunnel was completed in 1930 but its inauguration has been held up pending the completion of the rest of the line.

Waterworks Problems Discussed at Illinois AWWA Meeting

(Continued from p. 550)

ing materials. Sections of cast-iron pipe jointed with these materials were tested under various pressures. Lead pushed out, but with other materials failure occurred in breakage of plugs or caps at high pressures much above that for which the pipe was designed. Care to eliminate grease or oil, to use dry braided hemp and to keep the material within the narrow temperature limits set forth in instructions were emphasized.

A paper on the 1933 outbreak of amoebic dysentery in Chicago by Joel I. Connolly and Arthur E. Gorman contained slight information on the outbreak but discussed at length the hazard of old plumbing, faulty fixtures and cross connections to other sources of supply or to sewers. For 19 years the Chicago bureau of engineering has warned about the dangers of cross connections. At present an intensive inspection is being made of every hotel in Chicago by 56 sanitary engineers and plumbers. Ultimately it is intended to inspect all industrial plants, office buildings, hospitals and other structures housing large groups.

K. H. Logan, U. S. Bureau of Standards, contributed a paper on the selection of pipes for water distribution in the East Central part of the country based on the corrosion standpoint. Results of burying ferrous pipes twelve years ago in numerous places were outlined but conclusions cannot be drawn without all factors noted such as physical condition of soils, moisture, salt and acid content. Paul Hansen cited the use of a lime and sand backfill around the pipes laid in the filled ground at the Century of Progress. Certain pipes in or near buildings not so protected rusted out almost immediately.

Newly elected officers are: L. I. Birdsall, chairman, W. W. Polk, vice-chairman and A. E. Skinner, secretary-treasurer.

Code Authorities Start Functioning

(Continued from p. 550)

Presented with the question under the provisions of Section 10, Article VIII as to whether a contractor must specifically reserve the right to reject any or all bids in order to do so, it is stated in explanation No. 2 that nothing contained in Section 10 changes the contractor's status, nor deprives the contractor of any existing right to reject any or all bids. It is explained further that the provisions of this section do not make it mandatory on the contractor to subcontract with one of the mason contractors (as it happened to be in this case) from whom bids were invited, but that the contractor has the right to either do the work himself, or await the expiration of ninety days and invite new bids.

The authority was asked whether an awarding authority under Section 11 of Article VIII could require each general contractor to submit his list of subcontractors at the time of submitting his bid. In explanation No. 3 it is explained that under the code an awarding authority may require the naming of subcontractors only after the opening of bids.

Explanation No. 4 states that bids received from uninvited general contractors must be returned unopened. The question has been asked whether this provision contained in the third sentence of Section 7 of Article VII, applied to general contract bids as well as bids from subcontractors.

Inquiry was made by a manufacturing company furnishing material required for a construction project, whether it was satisfactory to obtain a list of general contractors bidding on a project, and furnish such bidders with a bid, whether the contractors actually solicited such bid or not. It was stated that many contractors would not bother to send cards of inquiry, but relied upon the manufacturers to ascertain that they were bidding, and supply them with a figure. It is stated in explanation No. 5 that the facts as given above do not constitute an invitation to bid.

In explanation No. 6, the authority states that nothing contained in Section 7 of Article VII prevents a general contractor from extending a general invitation for bids. The question asked was whether a general contractor must invite subcontractors in each specific case, or may extend a general invitation such as a letter to a list of subcontractors, saying that "in future when you note we are preparing a general bid, please accept this as an invitation for you to send us your estimate."

Settlement Made in Damage Suit Over Ohio State Office Building

Payment of $450,000 by the Columbus Gas and Fuel Co. has settled the two-year suit of the State of Ohio against the company, which resulted from the explosion that wrecked the interior of the $6,000,000 state office building. The settlement was a compromise and resulted in the dismissal of the state's suit. This settlment will affect the scores of personal damage suits now on file against the gas company. The accident was described in *ENR* April 21, 1932, p. 597.

Bid Shopping After Award Ended by New PWA Order

In an effort to minimize the common practice of bid shopping for subcontracts on the part of contractors after the opening of bids, the PWA has issued an order signed by Administrator Ickes to all state engineers of the Public Works Administration.

The order reads as follows:

"Every contractor who bids upon a project financed in whole or in part by loans or grants from the PWA shall submit in a sealed envelope with his bid to the contracting authority the names of all subcontractors and their bids upon which his bid is based. The sealed envelope so submitted shall have on it the name of the contractor with the words thereon 'Bids of Subcontractors.' Such submission shall be deemed to constitute an acceptance by the contractor, if awarded the contract, of the bid of each subcontractor. Any alteration therein, after the award of the contract, shall be subject to the approval of the contracting officer of the federal department or agency concerned."

Seattle Sells Bonds To Complete Skagit River Power Project

Permitting the immediate resumption of construction work on the Skagit River hydroelectric project, the city of Seattle has approved the sale of $4,500,000 of municipal light department bonds. On the recommendation of Supt. J. D. Ross, the city council accepted a bid made by a syndicate of bankers and bond dealers offering to take the $4 500,000 issue of 6 per cent bonds at par with a 10-day option on an alternate plan for bonds with 5 or 5½ per cent coupons, discounting to a 6 per cent basis.

Proceeds from the sale will (1) retire the $1,700,000 of light department construction warrants and interest, (2) complete the Diablo power plant and (3) finish the transmission line from the plant to the city.

Construction League Will Meet In Washington Next Week

A meeting of the general assembly of the Construction League of the United States will be held at Washington on April 30 and May 1. The establishment and affiliation of state construction leagues will be a primary feature of the program of discussion, and it is expected that applications for election to the parent body on the part of several state leagues will come before the meeting. Enlargement of the policy committee and of the secretarial organization will also be considered.

Personals

OLAF LAURGAARD, for 16 years city engineer of Portland, Ore., and since Jan. 1 in private consulting practice in that city, has been appointed by the bureau of reclamation as construction engineer on the Parker dam to be built by the bureau for the Metropolitan Water District across the Colorado river near Parker, Ariz., at the intake of the Colorado river aqueduct. Prior to his service as city engineer of Portland

Mr. Laurgaard had been active in reclamation and power development in the Northwest and had served with the Reclamation Service in its early years.

EDWARD HERING, formerly assistant engineer, has become acting chief engineer of the United States Pipe & Foundry Co., succeeding T. P. Anthony, chief engineer, recently resigned.

FRANK W. GARRAN, has been elected dean of Thayer School of Civil Engineering at Dartmouth. Dean Garran has been at Dartmouth since 1929 and succeeds Raymond R. Marsden.

E. J. PAULUS has been elected vice president of the McClintic-Marshall Corp., in charge of operations, succeeding the late Col. E. A. Gibbs. Mr. Paulus was formerly manager of the Chicago District for the company and has been with the company since 1912, serving in various capacities in the engineering, contracting and sales departments. ROBERT MACMINN has been appointed manager of the Chicago district, in charge of operations and sales. Mr. MacMinn has been with the company for more than 25 years in engineering capacities, including supervision of preliminary work on the George Washington bridge for the company.

Obituary

LEONARD E. BROWNSON, Havana, Cuba, president of Purdy & Henderson Co., well known American construction organization, died April 19, age 66. He had been in charge of the company's operations in Havana since 1899 and supervised the construction of many important buildings in Havana, including the National Capitol, the National Hotel of Cuba, and the Royal Bank of Canada in Havana. Prior to his joining the Purdy & Henderson Company, Mr. Brownson had worked for the George A. Fuller Co. He became president of the former organization in 1918.

ROBERT C. O'BRIEN, age 55, died in New Orleans April 19. For the past 12 years he had been a partner with Richard Sharp in the O'Brien Construction Co.

S. E. CRAIG, testing engineer for the Hydro-Electric Power Commission of Ontario for ten years, died in that city.

GEORGE H. T. SHAW, consulting engineer, died at New Orleans, La., on April 17, age 65 years. He had been inactive for some time on account of ill health. Mr. Shaw was engaged in railway survey work in this country in the early nineties and in 1892 was on location of a line from Buena Ventura, Colombia, over the Andes into the Canca River valley. Subsequently he was in private practice at Dixon, Ill., for several years and about 1903 went to Mexico as chief engineer of the F. C. Toluca, Balsas & Pacific Ry. Following the downfall of the Diaz government he returned to the United States, but continued his interest in Mexican affairs and returned there frequently for consulting work. For the past few years, until his health failed, he was with the bridge department of the Illinois state highway system.

CONSTRUCTION STATISTICS OF THE WEEK

ENGINEERING AWARDS at $23,985,000 are slightly above the average week to date but represent a considerable drop from last week's record total. The first four months total, $404,238,000, is 54 per cent ahead of the corresponding period last year. During the current week state and municipal awards held up well, $16,429,000, while federal awards dropped to $3,597,000. Private contracts at $3,959,000 are ahead of last week.

Highways, chiefly responsible for the state and municipal total, are $10,636,000, the second highest week of the year in this classification. Sewerage at $1,379,000 and bridges at $2,980,000 are well above last week. Public buildings dropped from their record total last week to $3,454,000 this week.

Large contracts for the week include: two 6-story apartments in Brooklyn, N. Y., $1,000,000; warehouse addition for Treasury Dept., Washington, D. C., $1,176,000; highway lettings, $708,000 by Alabama, $717,000 by Pennsylvania, $900,000 by Wisconsin, $1,001,000 by New York, $749,000 by South Dakota, $545,000 by Washington; sewer and water improvements in Greenville, S. C., $456,000; bridges by Illinois, $1,061,000; and reconstructing Wellington drawbridge between Medford and Somerville, Mass., $594,000.

Federal allotments by PWA this week were exceeded by rescissions and changes from loans and grants to grants only. This trend continues to release federal funds, earmarked for "loans," for reallotment.

CONTRACTS

(Thousands of Dollars)

	Weekly Average Apr. 1933	Prev. 4 Weeks	Week Apr. 26 1934
Federal Government	$1,756	$6,389	$3,597
State and municipal	5,060	15,432	16,429
Total public	$6,817	$21,821	$20,026
Total private	5,531	4,781	3,959
Week's total	$12,348	$26,602	$23,985
Cumulative to date: 1933.....	$263,232	1934.....	$404,238

NEW PRODUCTIVE CAPITAL

(Thousands of Dollars)

1934	Week Apr. 26	Cumulative
State and municipal....	$9,539	$103,975
PWA allotments, S&M..	—4,624	192,182
Corporate issues	....	13,369
PWA allotments, private	....	45,131
Total, Non-Federal....	$4,915	$354,657
PWA allotments Federal Constr.....	915	49,307
Total new capital.....	$5,830	$403,964
Cumulative to date: 1933.....	$72,986	1934.....	$354,657
Rescinded or reduced allotments (from 1933 PWA totals).....			$43,323

NOTE: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities, including the special highway funds; PWA private loans, and allotments for Federal construction.

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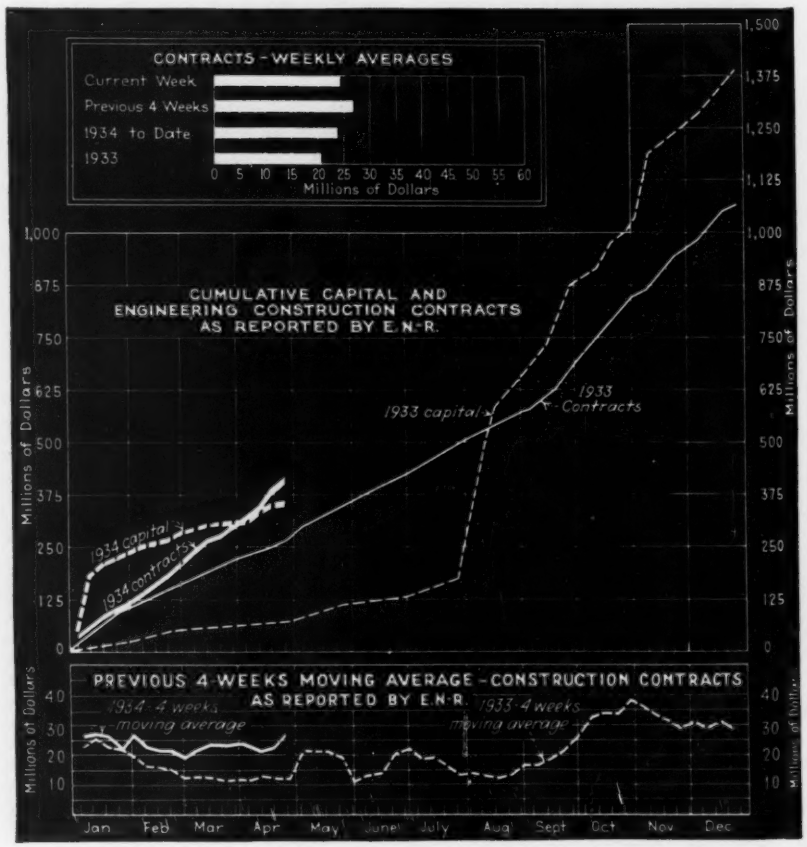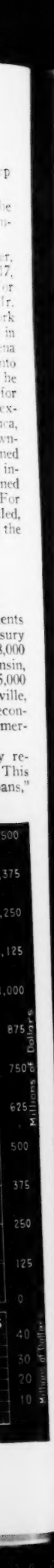
E.N.-R.-Cost		E.N.-R.-Volume	
April, 1934.....	195.86	March, 1934.....	109
March, 1934.....	194.04	February, 1934...	88
April, 1933.....	160.16	March, 1933.....	62
1933 (Average)...	170.18	1933 (Average)...	102
1932 (Average)...	156.97	1932 (Average)...	127
1931 (Average)...	181.35	1931 (Average)...	220
1913 (Average).....	100